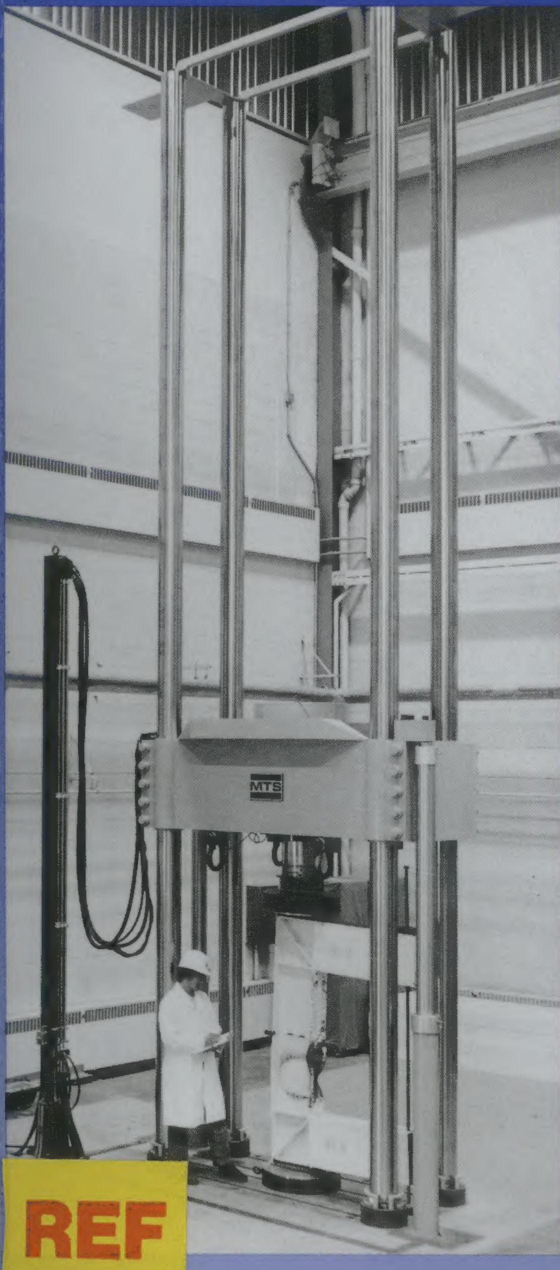

35 years of Structural Engineering

A Symposium in Honour of

D.J.L. Kennedy
G.L. Kulak
J.G. MacGregor
D.W. Murray
S.H. Simmonds
J. Warwaruk

[Un]iversity of Alberta
[De]partment of Civil Engineering

35 years of Structural Engineering
University of Alberta
Department of Civil Engineering

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June 24, 1994
Department of Civil Engineering
University of Alberta
Edmonton, Alberta

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FOREWARD

The University of Alberta underwent considerable expansion starting in the late nineteen-fifties and continuing through the sixties. This coincided with the transition of the University from a primarily undergraduate teaching institution to a major research oriented university. The Department of Civil Engineering shared in the University's growth during this period, with the addition of a number of new faculty. Of those hired in the structures area, many were to remain at the University of Alberta for their full academic careers. Thus began an era during which the Structures Group earned a national and international reputation for contributions in research, undergraduate and graduate education, and service to the engineering community and the community at large. This was due in large part to the individual and collective efforts of six members of the group: D.J.L. Kennedy, G.L. Kulak, J.G. MacGregor, D.W. Murray, S.H. Simmonds, and J. Warwaruk. Of these six, all but G.L. Kulak retired during the period 1992 to 1994.

In recognition of the significant achievements of these six men, a one-day Symposium was held in their honour on June 24, 1994. Attending the Symposium were many practicing engineers, colleagues, students, friends, and family members. A distinguished group of their colleagues and former teachers, drawn from across Canada and the US, presented the nine papers that comprised the technical program.

It is the wish of the Symposium Organizing Committee and the Department of Civil Engineering that this volume serve not only to record the technical program of the Symposium, but to document the careers of those being honoured and be a lasting tribute to their outstanding achievements. We dedicate this volume to them.

T.M. Hrudey
editor

SYMPOSIUM ORGANIZING COMMITTEE

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PRELIMINARY

A BRIEF REVIEW OF STRUCTURAL ENGINEERING AT THE UNIVERSITY OF ALBERTA

Jack Longworth
Professor Emeritus
Department of Civil Engineering
University of Alberta

The Department of Civil Engineering at the University of Alberta was founded in 1909, one year after the opening of the university. From the very beginning, Structural Engineering has been a significant component of the Civil Engineering curriculum. In the original curriculum statement, the third year program contained a lecture course on Theory of Structures, and a Testing Laboratory which included tests to illustrate the materials of construction. The fourth year program included a Design course in which students were required to complete a design of a truss bridge for a given loading and span; and, in addition, there was an optional course in Structural Engineering.

In 1912, I.F. Morrison, a graduate of M.I.T., became the fourth member to be appointed to the Civil Engineering staff and taught the structures courses in third and fourth year. In the succeeding years he was instrumental in shaping the development of the structural engineering curriculum and was an active member of staff until his retirement in 1954. Morrison developed the first postgraduate courses in structures at the University of Alberta. He served on various technical committees, including the committee which produced the first edition of the National Building Code of Canada, and acted as consultant on various municipal projects for the City of Edmonton and hydro-power projects for Calgary Power (now TansAlta Utilities). He also published a number of technical papers; for example, the 1945 annual Departmental report indicates he authored two papers and two discussions in that year. He obviously set a high standard for staff members who followed him in the field of structures.

In 1930, R.M. Hardy joined the Civil Engineering Department and he and Professor Morrison handled the duties in Structural Engineering. Their interest in foundation engineering design led to the introduction of the first course in soil mechanics taught at a Canadian university. The number of staff remained constant into the 1940's as engineering student enrolments actually decreased in the 1930's. However, beginning with the veterans' classes after World War Two, engineering enrolments increased dramatically and by 1947 the structures group had increased to four members. By 1960 Sid Simmonds, Jim MacGregor, Dave Murray and Joe Warwaruk had joined the group and they played leading roles in the development of the structures program. With improved research facilities and a large selection of courses the number of graduate students steadily increased. By 1970 the structures program at the University of Alberta was considered to be one of the best in the country. Geoff Kulak joined the group in 1970 and Laurie

Kennedy joined in 1982, bringing further expertise and making significant contributions to the structures group.

Postgraduate courses in structural engineering were introduced in the 1940's. The first two M.Sc. degrees in Civil Engineering with a structures emphasis were awarded in 1946 and the first Ph.D. in the structures program was awarded in 1965. In the 1960's, due largely to the efforts of members of the structures group, the first non-thesis Master's degree program at the University of Alberta was inaugurated. The Master of Engineering program in Civil Engineering has attracted practicing professional engineers desirous of upgrading and expanding their knowledge in the field of structures. To date a total of 270 graduate degrees have been awarded in the three degree programs in structural engineering.

Until the opening of the present Civil Engineering building in 1954, the Department was housed in the South Lab building where facilities available for experimental research in structures were very limited — the major equipment being a 75-ton capacity Riehle screw-type testing machine. In the new building improved facilities included an hydraulic universal machine with a capacity of 880 kips. However, it was not until the present Structural Engineering Laboratory was opened in 1964 that a truly adequate structural facility was available. At that time it was a state-of-the-art facility; one of the best in Canada. Over the intervening years additions and improvements have maintained that high standard. The large area test floor on which loads are applied by jacks reacting on movable steel frames provides considerable flexibility, and accommodates a wide range in size and shape of test specimens. A separate closed-loop fatigue loading facility is also available. In addition two electro-hydraulic testing machines can apply static or slowly applied cyclic loads in tension or compression. Computer controlled data acquisition equipment allows for large volumes of test data to be recorded and processed and a skilled technical staff assures that projects are efficiently organized in this extremely busy facility.

The structural engineering program at the University of Alberta is a well-balanced program including sixteen graduate courses. The expertise of the structures group covers a wide spectrum of research activities based on analysis and experimentation related to structural behavior and design. Individual staff members specialize in analysis, reinforced concrete, prestressed concrete, steel, masonry and timber structures. Results of this research are reported in journals and at conferences and in the past five years the members of the group have published 75 papers in refereed journals and 64 papers in conference proceedings.

The group maintains an effective relationship with the concrete, steel, masonry and timber industries. Research projects have involved co-operation with agencies such as the Alberta Research Council, the Centre for Frontier Engineering Research, the Federal Department of Forestry, the Canadian Institute of Steel Construction, the Canadian Institute of Timber Construction and the Prairie Masonry Institute.

Involvement in technical committees has always been an important activity for members of the structures group. Of special focus are committees relating to building standards and regulations. These include committees related to the National Building Code of Canada - the Associate Committee on the National Building Code and the

Standing Committee on Structural Design; together with Canadian Standards Association committees dealing with various material design standards.

In 1975 the structural laboratory was named the I.F. Morrison Structural Engineering Laboratory in honor of the pioneer in structural engineering at the University of Alberta. The name is a fitting reminder of the priorities set by Professor Morrison and continued through the years by the structures group.

CHESTER P. SIESS

SAVOY, ILLINOIS

28 April 1994

GREETINGS

To: Joe Warwaruk
Jim MacGregor
Laurie Kennedy
Sid Simmonds

The 1950s and 1960s were exciting times in Urbana-Champaign. In Talbot Lab, in Urbana, we were doing pioneering research on prestressed concrete and on multiple-panel floor slabs. The superb facilities in Talbot Lab were matched by the luxurious and spatial accommodations provided for the research assistants in Structural Research Annex B, across the street in Champaign. And it was there that we had some of the finest Research Assistants I ever had the pleasure and challenge of working with.

Joe Warwaruk arrived from Alberta in 1955, soon to be followed by Jim MacGregor in 1956 and Sid Simmonds a little later in 1960. I have always felt certain that their choice of Illinois was inspired or encouraged by Ralph McManus, who had returned to Alberta in 1952. Laurie Kennedy came back to Illinois from Toronto in 1958. I was fortunate to have all of them in my graduate courses on behavior and to have them do their theses under my direction.

All of these men returned from whence they came after receiving their PhD degrees, and all chose teaching as a career. I am sure they know, as I do, that the greatest reward a teacher can have comes from the accomplishments and successes of his students. But this reward is multiplied many times when his students themselves become teachers and pass on to another generation some of the things they have learned from him.

I have been fortunate to have had many fine students whose successes in their profession have made me proud indeed, but none more than these four men who are now retiring from the Alberta faculty. I wish them happiness and success in all they now plan to do, whether it be work or play or relaxation. May their retirement be as enjoyable and satisfying as mine has.

Helen joins me in sending our sincere best wishes,

Chet

THE HONOUREES

David James Laurie Kennedy

A Personal History

Laurie was born in February 1929 in the little agricultural village of Cumberland, nestled in the Ottawa valley, about 16 miles east of Canada's capital. Eight months later, almost to the day, in October the stock market collapsed. No connection has been established between these two events, and, in any case, the second has proven more significant.

Why was Laurie given so many names and why is he called by his third? He was the fourth son and the fifth of the six children of Robert James and Robena Eva, and none of the previous sons had been named after either of the grandfathers who lived close by. Therefore DJL got both his "Grampa's" names, but fear of offending one or the other required a third name - Laurie - a good Scottish name, although sometimes embarrassing when one is expected to be of the opposite sex. Actually, through his childhood he was known by another that is still used by his siblings and early school friends. When calling with his friend Fred Jones at the manse of the Reverend Campbell and, being unknown to the latter as he was not a member of Campbell's flock, he introduced himself as "Huckleberry Finn". The name stuck and anyone from Cumberland knows him as Huck.

Laurie attended Cumberland public and continuation school, that's right, it was one school - really big with four rooms and four teachers. Somewhere in public school he skipped a grade or two and following the first of these occasions, in the ensuing Christmas exams did not fare too well. Presentation of his report card *with numeric grades and class rankings*, brought a mild admonishment from his father "Laurie, you can do better than that." There is a moral or two there, one for educators and another for children. The school's finances were somewhat limited which meant optional courses did not exist. Thus Laurie had to take Latin for three years. He considers this was invaluable in enhancing language skills as well as in developing disciplined thinking.

Life in the little pastoral community was simple. Summertimes were barefoot days with a pair of short pants as the only clothing. This had at least one unknown advantage, when the summer evenings grew cool the skinny shivering children ran home. Days were often spent swimming in the Ottawa river, probably more than slightly polluted, which may explain why few children were ever sick. By the time they got to their teens

they had been exposed to every virus and bacterium known to medical science. Other times were spent roaming the nearby hills, building tree forts from old boards and discarded Coke signs or having innumerable "wars" as the group of about 15 formed two sides, which were, of course, different the next time. Weapons were often mullein stocks used as javelins or spears. Winter brought winter sports, the chief of which was playing "road-hockey". The puck was frequently a frozen horse-ball.

Laurie spent the final year of Ontario's thirteen-year primary and secondary system at Lisgar Collegiate Institute in Ottawa, travelling to and from the city each day on crowded foul smelling buses. He would be lucky to get a seat for the last mile home.

His first year as a student at the University of Toronto was spent in Honours Science where he managed to get first class honour standing. However, now at the ripe old age of 18 he decided he didn't want to follow a career in biochemistry or anything like it. He switched to Engineering and, on the advice of the Assistant Dean, so that he would be eligible for scholarships, took the full programme without any advanced standing credits. This worked, or perhaps Laurie did, and through the four years of the programme he topped Civil Engineering every year. In the third year he was top student in the Faculty and netted the really BIG award which was worth about $300.

At graduation in 1951, he had the option of an Athlone Fellowship or a graduate scholarship at the University of Illinois and took the latter. Marriage followed on the heels of graduation and by the time of his return to Ottawa from Illinois in June 1952 he had a new American son, David, born two months premature, and an MS in Civil Engineering.

Laurie worked in the Division of Building Research for 10 months but jumped at the chance to join the Soil Mechanics group of the Foundation Company of Canada in Montreal. There, son #2 Michael arrived in February 1954 while Dad was doing a soils investigation (we didn't know the word geotechnical in those days) for a couple of bridges on Highway 17 on the north shore of Lake Superior. Boy was it cold, with the wind blowing in off the lake. It was brass monkey weather, often dropping to -40C. They worked 9-hour days. On arriving at the drilling site in the morning the first thing was to light the kerosene-filled 'salamander' and place their sandwiches around the stack. The foreman would then heat the oil in the crankcase of the drill rig with a blowtorch. Five hours later, at noon, the four of them, the foreman, two labourers and the engineer, would be ravenously hungry. The sandwiches were there, but in spite of sitting on the salamander would be partly frozen, with the added advantage of tasting of kerosene. The crew worked 54-hour weeks, but the engineer had daily reports to mail back to Montreal each night. This meant another hour or two. With all of this the labourers got paid the most, the foreman next and the salaried professional, with no overtime, the least. The engineers in the Soils group put in much overtime with time sheets of around 200 to 220 hours/month (most of it billable) compared to a basic month of 162.5 hours. However, Xmas bonuses were promised. But these never came because another segment of the company may have not done very well. Not to worry though, there was always the prospect of *huge* annual salary increases. One of $25/month is remembered in particular.

The firm's name was changed to Geocon Limited (standing for geotechnical construction, they now knew what the word meant) and in 1955 or thereabouts Laurie was moved to Toronto to take charge of that office. The family bought their first house, costing all of $13 000. Two things happened in the fall of 1956. Stephen, son #3, arrived. Almost at the same time Laurie decided to look for a job as a structural engineer as that was what he really had wanted to do since his undergraduate days.

Taking a week's overdue vacation he lined up more than a dozen jobs one of which was a sessional lectureship at the U of T, with a guarantee of employment only to the end of April. He also had the chairman's promise that he would help find a summer job if DJLK proved satisfactory and IF the CE department wanted him back in September - when he would get another nine month appointment. The thought of even just teaching, there would be no research involvement as a sessional, was appealing. The pay cut was manageable and Kennedy was started on an academic career.

He liked it, including first year Statics, but soon realized that a Ph.D. was necessary for any degree of professional satisfaction in the academic world. Soon he approached the head of the department, Carson Morrison, and it was agreed that he would be given a two year leave of absence for Ph.D. studies at an annual salary of $1000, that's right, plus continuing accumulation of time to the pension fund. Laurie decided that he would work one more year as a sessional before heading back to Illinois where they knew him and where he had been offered a research fellowship to work, as it turned out, under Professors Peck and Seiss. During this time Laurie met Jim MacGregor who was introduced to Laurie's three sons Dave, Mike and Steve as the guy who had Peter Rabbit in his cabbage patch.

Two intense years of study and work followed until September 1960 when Laurie delivered the family home to save rent at the end of August. All their worldly goods were in the U-haul trailer and in the car were now four children. Carolyn had been born in December 1959 but didn't result in any income tax savings for that year for the very simple reason that there was little income. Laurie returned the trailer to Illinois to save the one -way fee and also to get a bit more work done.

He arrived back in Toronto on the Saturday before classes started. His Ph.D. was not yet complete. All the experimental work was completed and the data, including strain data laboriously recorded by hand, had been reduced. All that remained were the somewhat critical steps of analyzing the data and writing the thesis itself. Laurie, at some time must have been awfully wicked, because he got no rest and carried a full teaching load of about 18 hours/week. Work on the thesis lagged for about three months until a wake-up letter arrived from Prof. Seiss. By the end of March the thesis was done and sent off to Seiss. The defense was successful although Peck gave him some anxious moments. At U of T a promotion to the exalted rank of Assistant Professor followed.

Carson Morrison asked Laurie to work with his consulting firm, Morrison Hershfield Millman and Huggins and in 1962 he was invited to buy into the firm as a junior partner. This association continued for 20 years, only ending when Laurie resigned from the firm to join the U of A in June 1982. There was lots of fun and excitement and challenges in the design and construction world. Projects that Laurie

was responsible for included field review of construction of the CBC headquarters building in Ottawa, the Leaside bridge design and reconstruction in Toronto (he has lots of stories to tell about that) (the firm won two awards on this project) and the Air Canada Pavilion at Expo 67. The latter, 28 years ago, was designed in fibre-reinforced plastic; number one problem: low modulus of elasticity.

Work at U of T continued and research activity gradually picked up though the departmental emphasis, as you may have concluded, was more, with the exception of Mark Huggins, on professional practice, otherwise known as consulting. In October 1962 the fourth son and fifth child, Timothy was born, completing the complement of children in the Kennedy home.

Laurie's long and happy association with the Canadian Institute of Steel Construction began at about this time when Don Turner asked him to give a series of evening lectures on "Modern Steel Structures". The course developed as 20 lectures, one evening per week lasting two hours or more. The course was very successful and over a period of five years or so was taken by about 400 practicing engineers from the Toronto area. Why even designing bracing for both strength and stiffness, only now introduced in S16, was discussed. Laurie got the first CISC research grant and with Bill Rowan did work on open web steel joists.

Also in 1962 Laurie became a member of the S16 Committee of the Canadian Standards Association. This began Laurie's work on structural design committees which has continued to this day. DJL became chairman of the S16 committee in 1968 in time to produce the last working stress design standard for steel structures in Canada. (All steel buildings are required to be designed in accordance with such a standard.) In 1974 the committee introduced the first Limit States Design Standard for Steel Structures in the western world, 12 years before the Americans. Laurie also chaired the LSD committee for steel highway bridges in the OHBDC (Ontario Highway Bridge Design Code) and that in the Canadian Code which has followed it. He chairs the fixed steel offshore structures code too. His code activities are international in scope with work on ASCE and ISO committees. This work is generally related to limit states design. He serves on other CSA and National Building Code committees as well.

With five children, a new larger home was required. This took the form of a new five-bedroom house in Don Mills. Later a 42-foot swimming pool was built in the backyard and Laurie decided that a deck, small retaining walls and patio with steps to and fro, made from local Credit Valley sandstone would be in order. Many tons of stone later (tonnes didn't exist then) it was beautiful (unbiased opinion). The boys all pitched in to help Dad. Even wee Tim, about four then, would hand his Dad the stones he had cut and which had been numbered for placing in the retaining walls. Only once did Tim make an error when he brought stone #63 instead of #36. One Saturday morning while Laurie was cutting stone, the garden gate was opened. Laurie, thinking it was a neighbour and working on a difficult cut continued to work without looking up. When the cut was finished he was surprised to see a stranger, probably attracted first by the large pile of stone at the curb and then the noise of chipping stone, who then said "I'll give you $18 an hour to start work on Monday." That was a lot more than Laurie was making at the U of T.

In 1970, Laurie joined Carleton University as Professor and Chairman of the Division of Solid Mechanics and Structural Engineering (Carleton's equivalent to Civil Engineering). While there he worked closely with Don Scott who was chairman of the CE Department at the U of O.

Out of a clear blue sky came a request to apply for the job of Dean of Engineering at the University of Windsor. Laurie said "why not" and the year 1973 saw Laurie moving to Windsor. Three major challenges faced him, the least of which was a complete revision of the undergraduate curricula to accommodate seven programmes with a minimum number of staff. Undergraduate enrollment was down and decreasing. A vigorous and successful high school interaction programme was undertaken into which faculty members jumped with both feet. Enrollments moved back up to about 220 in the first year. The Engineering Faculty's Ph.D. programme had been given a black eye in a report to the Ontario Council of Graduate Studies. About three years later it got a clean bill of health. Sometime in here a marriage became unglued, perhaps the less said about that the better.

The University of Melbourne beckoned with a visiting professorship which resulted in Laurie's very first sabbatical. It was made much more pleasant by a two month visit by his daughter Carolyn who accompanied him to Australia. In a little white Datsun nicknamed "The Great White Shark" they saw much of the south and east of Australia including the vineyards in the Borassa valley. Tim also made his father's day when he joined his Dad for a stretched spring break. Together they flew in a rented Cessna 182-RG to Alice Springs and Ayer's Rock, a marvelous monolith rising out of the desert. Dad had a hard time keeping within hailing distance of Tim as they climbed to the top of the rock.

While in Australia, Laurie delivered a series of one-day seminars on limit states design all over the continent, Melbourne, Adelaide, Perth, Hobart, Sydney, Newcastle and Brisbane. DJLK also addressed, at the invitation of Max Lay, an assembly of state highway engineers at the Australian Road Research Board, about the new OHBDC which was of course in limit states format.

Returning to Canada in 1980, Laurie soon surfaced in Edmonton where he took charge of the Edmonton office of his old consulting firm. Things looked rosy for a while, but then the National Energy Programme came into effect and the consulting business took a nose dive. In the spring of 1982 he had two options: close up shop and return to the firm's HQ in Toronto or else resign from the firm (paying capital gains tax like mad on his shares) and join the excellent structures group at the U of A which just happened to be looking for a man of steel. Happily for him he took the latter course, renewed old friendships with Geoff, Pete and Jim whom he knew from his committee work and made others. He considers these Alberta days to have been among the happiest and most productive of his career.

His code work has taken him to the UK, France, the Soviet Union, Germany, Sweden, China, Norway, Denmark, Yugoslavia and South Africa. The PUC connection has taken him to Brazil many times. He only needed three trips to Brazil to finally meet and snare Therezinha (TARE-EH-ZEEN-YA). They were married in 1991, in Connecticut of all places, with arrangements made by The's daughter Laura and Laurie's

son Steve. Between them, The and Laurinho have nine children. They are now watching grand children appear. Apparently it is magic, they now have about six and eight-ninths.

Laurie has been much influenced by his family. Robert, his father, was clerk of Cumberland Township for some 35 years and an honourable man who was determined that his children would have the advantages of higher education. His mother raised six children while operating a maternity home as an RN where over 400 children were born in this same period. Laurie's brothers are: Robert, retired as chief Librarian for the Bell system in the USA, Irving retired from general medical practice in rural Ontario and Ontario's leading fighter pilot in WWII with 14 enemy aircraft to his credit, and Carleton killed on August, 30, 1944 in the RCAF. His sister Evelyn, a retired flight attendant with Delta, lives in Atlanta. His kid sister Joyce is a university administrator (one of the good effective ones) with California State University Northridge, Ventura Campus.

Laurie's five kids have 12 university degrees among them including four from the U of A and six in CIVIL engineering. One of them is a CE professor.

Laurie, in spite of his upbringing, is a free-thinker and is irreligious. He is an atheist. As a humanist, he sees no need to rely on supernatural beings to help him on his way through life. He attempts to live an unprejudiced and moral life. On his office door you will find this quotation from Junius "The subject who is truly loyal to the chief magistrate will neither advise nor submit to arbitrary measures."

Professional Career Summary

D.J. Laurie Kennedy, Ph.D., P.Eng., F.CSCE

Laurie Kennedy, a structural engineer, specializes in the behaviour of steel structures with a current emphasis on limit states design. He is recognized for his work on composite construction, bridge evaluation, welded connections, structural loads, structural stability and limit states design. His work on the development of building codes and steel design standards spans 32 years. This work encompasses Canadian, American and International standards where he has served on 23 committees and is still serving on 12. Currently, he is the chairman of three committees for the design of steel structures in Canada, those for steel buildings, steel bridges and steel fixed offshore structures.

Results of his research have been incorporated into steel design standards and loading codes for Canada, the United States, and internationally as well.

He served as fifth president of the Canadian Society for Civil Engineering in 1975-76.

EDUCATION

B.A.Sc.	Civil Engineering	University of Toronto	1951
M.S	Civil Engineering	University of Illinois	1952
Ph.D.	Civil Engineering	University of Illinois	1961

EMPLOYMENT HISTORY

Laurie Kennedy is Professor of Civil Engineering at the University of Alberta, Edmonton, Alberta, Canada. In addition to his university duties and his significant volunteer contributions on numerous standards and code committees he consults from time to time on problems related to the evaluation of structures and loads acting on them in answer to the question: Why did the structure suffer distress or how is the probability of distress minimized?

1952-1953	Research Officer, Division of Building Research National Research Council, Ottawa.
1953-1956	Jr. Engineer to District Soils Engineer, Foundation Co. and Geocon Limited, Montreal and Toronto.

1956-1970	Lecturer to Associate Professor, University of Toronto
1962-1982	Member to Director, Morrison Hershfield Burgess and Huggins, Consulting Engineers, Toronto, Ottawa.
1970-1973	Professor and Chairman, Division of Solid Mechanics and Structural Engineering, Carleton University, Ottawa.
1973-1979	Professor and Dean, Faculty of Engineering, University of Windsor, Windsor.
	President, Industrial Research Institute, University of Windsor, Windsor.
1979-1980	Visiting Professor, University of Melbourne, Melbourne, Australia.
1980-1982	Principal-in-charge, Morrison Hershfield Burgess and Huggins, Consulting Engineers, Edmonton, Alberta.
1982-1994	Professor, Department of Civil Engineering, University of Alberta.

PROFESSIONAL AND TECHNICAL ASSOCIATIONS

Laurie Kennedy is a registered engineer in Quebec, Ontario, and Alberta. He is a fellow of the Canadian Society for Civil Engineering, the American Society of Civil Engineers, and a member of the International Association of Bridge and Structural Engineers and of the American Concrete Institute. Current standards and code development work is related to the following committees: National Building Code of Canada, Standing Committee on Structural Design; Canadian Standards Association, Committee S16, Steel Structures for Buildings (Chairman); Canadian Standards Association, Canadian Highway Bridge Code (member and Chairman of Steel Bridge Committee); Canadian Standards Association, Committee S473, Steel Fixed Offshore Structures (Chairman); Canadian Standards Association, Committee S471 Environmental Loads and Design Criteria; Canadian Standards Association, Steering Committee for Offshore Structures; Canadian Standards Association, Steering Committee on Structures (Design) (Chairman); Canadian Standards Association, Committee on Limit States Design (Chairman); American Society of Civil Engineers, Committee 7, Minimum design loads for buildings and other structures, Subcommittee on load combinations; International Standards Organization, TC 167SC1 and TC 98SC3.

In addition to his year as President of the Canadian Society of Civil Engineers (1975-1976) he has served as chairman of the Honours and Awards Committee and of the Technical Activities Committee.

HONOURS AND AWARDS

1947-1950	Various undergraduate scholarships.
1951	Graduate fellowship University of Illinois.
1960	Sigma Xi; Phi Kappa Phi.
1967-1969	Leaside Bridge. Partner-in-charge. Cominco award for excellence in galvanizing; Canadian Consulting Engineer, First prize for civil engineering structures.
1977	Government of Canada Silver Jubilee Medal.
1980	Canadian Society for Civil Engineering, Gzowski Medal (with M.M. GadAly) for best paper in structural engineering.
1984	Canadian Standards Association, Award of Merit for contributions in development of standards.
1989	Canadian Society for Civil Engineering, AB Sanderson Award for outstanding contributions to the development and practice of structural engineering in Canada.
1989	Roads and Transportation Association of Canada, Presidents Award for best paper at annual conference.
1990	Canadian Institute of Steel Construction, Special Award for selfless contributions to structural engineering in Canada.
1992	Canadian Society for Civil Engineering, Gzowski Medal (with A. Brattland) for best paper in structural engineering.
1992	Canadian Society for Civil Engineering, P.L. Pratley Award (with F.M. Bartlett and P.G. Buckland) for best paper in bridge engineering.
1994	American Society of Civil Engineers, Shortridge Hardesty Award for outstanding contributions to the field of structural stability.

Some invited lectures seminars

1978	M.W. Huggins Symposium, Toronto, September.
1979	Australian Institute of Steel Construction, one-day seminars, Hobart, Melbourne, Adelaide, Perth, Brisbane, Sydney, Newcastle, October-November.

1980	The Colonel Edward Evans Memorial Lecture, University of Queensland, Brisbane, May.
1985	The 1985 International Symposium on Structural Steel, American Institute of Steel Construction and Canadian Institute of Steel Construction, Chicago, May.
1986	Projeto nos Estados Limites. 2° Colloquio de estruturas de aço na PUC/RJ, Julho.
1986	South African Institute of Steel Construction. Seminars on future developments in limit states design, Durban, Johannesburg, Cape Town, Pretoria, Stellenbosch, August.
1988	Anais do Simposio Brasileiro de Estruturas de aço, Pontificia Universidade Catolica do Rio de Janeiro, Rio de Janeiro, Julho.
1989	PUC/RJ, Visiting Professor, March-June.
1989	First International Conference on Structural Engineering. Ain-Shams University, Cairo, May.
1990	South African Institute of Steel Construction. Seminars and lectures, Durban, Johannesburg, Cape Town, Pretoria, Stellenbosch, August.
1991	Norwegian Technical Institute, Trondheim, September.

PUBLICATIONS

Laurie Kennedy has had 81 papers published in refereed journals or conference proceedings. He has contributed to two books and has over 100 scholarly addresses and technical reports to his credit. Contributions to books and papers in referred journals or published conference proceedings for the last six years are:

Books

Mikhailovsky, L. and Kennedy, D.J.L., 1993. Appendix B, Flexural behaviour of bolted connections in Soil-steel bridges, Design and Construction, G. Abdel-Sayed, B. Bakht and L.G. Jaeger, McGraw Hill, Toronto.

Kennedy, D.J.L., 1993. Composite trusses in Chapter 3, Composite beams, of Composite Design for Buildings. American Society of Civil Engineers, Editor I.M. Viest, submitted.

Refereed Journals

Essa, H.S. and Kennedy, D.J.L., 1994. Design of cantilever beams: a refined approach. Journal of Structural Engineering American Society of Civil Engineers. Accepted.

Kennedy, D. J. L., Kulak, G. L. and Driver, R. 1994. Discussion of Post Buckling Behavoiur of steel-plate shear walls under cyclic loads. M. Elgaaly, V. Caccese and C. Du. Journal of Structural Engineering. American Society of Civil Engineers, Vol. 119, No. 2. Accepted.

Kulak, G. L., Kennedy, D. J. L. and Driver R. 1994. Discussion of Experimental study of thin steel-plate shear walls under cyclic load. V. Caccese, M. Elgaaly and Ruobo Chen. Journal of Structural Engineering, American Society of Civil Engineers, Vol. 119, No. 2 Accepted.

Essa, H.S. and Kennedy, D.J.L., 1994. Station square revisited: distortional buckling collapse. Canadian Journal of Civil Engineering, Vol. 21, June.

Lu, Y.Q. and Kennedy, D.J.L., 1994. The flexural behaviour of concrete-filled hollow structural sections. Canadian Journal of Civil Engineering, Vol. 21, February, pp. 111-130.

Essa, H.S. and Kennedy, D.J.L., 1993. Design of steel beams in cantilever-suspended span construction. Journal of Structural Engineering American Society of Civil Engineers, submitted July.

Lu, Y.Q. and Kennedy, D.J.L., 1993. Correction of residual strain for coupons that curve on sectioning. Canadian Journal of Civil Engineering, Vol. 20, August, pp. 712-714.

Kennedy, D.J.L., Picard, A. and Beaulieu, D., 1992. Limit states design of beam-columns: the Canadian approach and some comparisons, Journal of Constructional Steel Research. Vol. 25, pp. 141-164.

Bartlett, F.M., Buckland, P.G. and Kennedy, D.J.L., 1992. Canadian highway bridge evaluation: Derivation of Clause 12 of CAN/CSA-S6-88. Canadian Journal of Civil Engineering, Vol. 19, pp. 1007-1016.

Kennedy, D.J.L., Gagnon, D.P., Allen, D.E. and MacGregor, J.G., 1992. Canadian highway bridge evaluation: load and resistance factors. Canadian Journal of Civil Engineering, Vol. 19, pp. 992-1006.

Kennedy, D.J.L. and Brattland, A., 1992. Reply to discussion by M. Sarkisian and H. Iyengar on Flexural tests of two full-scale composite trusses. Canadian Journal of Civil Engineering, Vol. 19, pp. 930-931.

Albert, C., Essa, H.S. and Kennedy, D.J.L., 1992. Distortional buckling of steel beams in cantilever suspended span construction. Canadian Journal of Civil Engineering, Vol. 19, pp. 767-780.

Mikhailovsky, L., Kennedy, D.J.L. and Lee, R.W.S., 1992. The flexural behaviour of bolted joints of corrugated steel plates. Canadian Journal of Civil Engineering, Vol. 19. pp. 896-905.

Picard, A., Beaulieu, D. and Kennedy, D.J.L., 1992. Longuer de flambement des éléments en compression. Construction Métallique CTICM Saint-Rémy-lès Chevreuse, France, Vol. 29, No. 2, pp. 3-15.

Kennedy, D.J.L. and Brattland, A., 1992. Shrinkage tests of two full-scale composite trusses. Canadian Journal of Civil Engineering, Vol. 19, pp. 296-309.

Brattland, A., and Kennedy, D.J.L., 1992. Flexural tests of two full-scale composite trusses. Canadian Journal of Civil Engineering, Vol. 19, pp. 279-295.

Kennedy, D.J.L., Picard, A., Beaulieu, D. and Andrade, S., 1991. Discussion of Design interaction equations for steel members by I.S. Sohal, L. Dunn and W.F. Chen. Journal of the Structural Division, ASCE, Vol. 117, ST 7, pp 2189-2191.

Chernenko, D.E. and Kennedy, D.J.L., 1991. An analysis of the performance of welded wide flange columns. Canadian Journal of Civil Engineering, Vol. 18, pp. 537-555.

Kennedy, D.J.L., Picard, A. and Beaulieu, D., 1990. New Canadian provisions for the design of beam-columns. Canadian Journal of Civil Engineering, Vol. 17, December, pp. 873-893.

Kennedy D.J.L., Miazga, G.S. and Lesik, D.F., 1990. Discussion of Evaluation of fillet weld shear strength of FCAW electrodes by R.W. McClellan, Welding Journal, August 1989. Welding Journal, pp. 44-46.

Lesik, D.F. and Kennedy, D.J.L., 1990. Ultimate strength of fillet welded connections loaded in plane. Canadian Journal of Civil Engineering, Vol. 17, February, pp. 55-67.

Miazga, G.S. and Kennedy, D.J.L., 1989. Behaviour of fillet welds as a function of the angle of loading. Canadian Journal of Civil Engineering, Vol. 16, August, pp. 583-599.

Gagnon, D.P. and Kennedy, D.J.L., 1989. Behaviour and ultimate tensile strength of partial joint penetration groove welds. Canadian Journal of Civil Engineering, Vol. 16, June, pp. 384-399.

Driver, R.G. and Kennedy, D.J.L., 1989. Combined flexure and torsion of I-shaped steel beams. Canadian Journal of Civil Engineering. Vol. 16, No. 2, April, pp. 124-139.

Published Conference Proceedings

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Essa, H.S. and Kennedy, D.J.L., 1993. Design of restrained beams against lateral-torsional buckling. Annual conference, Canadian Society for Civil Engineering, Fredericton, NB. June, 10 pages.

Essa, H.S. and Kennedy, D.J.L., 1993. Bracing in cantilever-suspended span construction. Annual conference, Structural Stability Research Council, Milwaukee, WI. April, 12 pages.

Kennedy, D.J.L., Isyumov, N. and Mikitiuk, M., 1992. The effectiveness of code provisions for snow accumulations on stepped roofs. Second International conference on Snow Engineering. Engineering Foundation Conference, Santa Barbara, CA, June 21-26, 14 pages.

Hosain, M.U., Chien, E.Y.L., and Kennedy, D.J.L., 1992. New Canadian provisions for the design of composite beams. Second International Conference on Composite Construction, Engineering Foundation Conference, Potosi, MO, June 15 - 19.

Lu, Y.Q. and Kennedy, D.J.L., 1992. The flexural behaviour of concrete-filled hollow structural sections, Annual Conference, Canadian Society for Civil Engineering, Québec, QC, May 26 - 29, 11 pages.

Essa, H.S., and Kennedy, D.J.L., 1992. Design of cantilever beams against lateral-torsional buckling. Annual Conference, Canadian Society for Civil Engineering, Québec, QC, May 26 - 29, 11 pages.

Shaker, A.F. and Kennedy, D.J.L., 1991. The effective modulus of elasticity of concrete in tension. Annual Conference, Canadian Society for Civil Engineering, Vancouver, May 29-31, 10 pages.

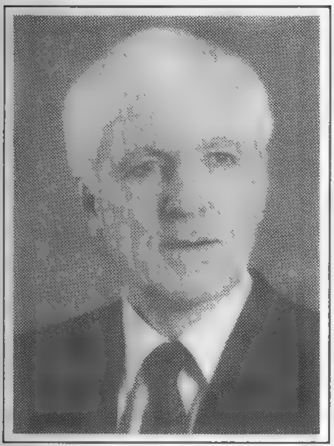
Albert, C., Essa, H.S. and Kennedy, D.J.L., 1991. Distortional buckling of steel beams in cantilever suspended span construction. Annual Conference, Canadian Society for Civil Engineering, Vancouver, May 29 - 31. 10 pages.

Kennedy, D.J.L., Grady, D.R. and Dozzi, S.P., 1991. Limit states design of wooden formwork for concrete walls. Construction Congress '91, American Society of Civil Engineers, Cambridge, Mass. April 13-16. 6 pages.

Kennedy, D.J.L., Beaulieu, D. and Picard, A., 1991. Effective length factors in the Canadian steel design standard. Structures Congress, American Society of Civil Engineers, Indianapolis, April 29 - May 1, 4 pages.

Kennedy, D.J.L. and Chernenko, D.E., 1990. An analysis of the performance of welded wide flange columns. Annual Conference, Structural Stability Research Council, St. Louis, MO., April, 11 pages.

Kennedy, D.J.L., 1990. Tests of two 38 foot span composite trusses. 1990 National Steel Construction Conference. American Institute of Steel Construction, Kansas City, MO., pp. 14-1-14-27.



Geoffrey L. Kulak

A Personal History

Geoffrey Luther Kulak was born in Edmonton in 1936, the last of the three children of John and Mary Kulak. His was educated in the public school system in Edmonton, and he entered the engineering program at the University of Alberta in 1954, graduating in 1958. Geoff's father was a school teacher, and both parents instilled a strong sense of the need for education, preferably in one of the professions, in their children. Both parents are still alive and living in Edmonton.

In May 1958, Geoff not only graduated from the engineering program, but married. Alice Long, whose father was professor of History at the University of Alberta, and Geoff had known each other during their high-school days (Strathcona High School), and strengthened that friendship at University. Following their marriage, Geoff was in the unusual position (certainly for that time) of supporting his wife while she finished her university degree, which is in History. Their family consists of two daughters, both graduates of the University of Alberta. Jennifer, who graduated in Art History, is married to a lawyer. They live in Calgary, and are the parents of Geoff and Alice's two grandchildren. Alison graduated in Honours Psychology from the University of Alberta, and then took degrees at the University of Saskatchewan (M.Sc. in Clinical Psychology) and Carleton University (Ph.D. in Cognitive Psychology). She is currently teaching at Wilfred Laurier University in Kitchener, Ontario.

A training ground for many of Alberta's structural engineers was the Bridge Branch of the Department of Highways of Alberta, and Geoff's first employment after graduation was with that group. Although this period of practical experience was relatively short (2-1/2 years), it was an excellent location to learn the realities of structural engineering, including the experience of seeing how designs were turned into real bridges. Because of the encouragement of some of his colleagues at the Bridge Branch who had attended graduate school, Geoff returned to university in 1960 and graduated with an M.S. degree from the University of Illinois in 1961. It was this exposure, especially the teaching of Prof. Edwin Gaylord, that first kindled his interest in the behavior of steel structures. At this period of time, the University of Illinois was the preeminent location for those who wanted to study the behavior of steel or concrete

structures. In the list of those six being honoured in the Symposium, only Dave Murray, whose area of specialization is structural analysis, does not have at least one of his degrees from Illinois.

The educational experience at Illinois convinced Geoff that he wanted to make a career in university teaching. Opportunities were not plentiful, but he was able to fill a one-year position at the University of Alberta, and this experience convinced him that he would enjoy the work. The one year at Alberta was also influential because he met and became friends with Peter Adams, who would later encourage Geoff to attend Lehigh University. They established a life-long friendship and collaboration in steel structures.

A teaching position opened up at Nova Scotia Technical College (now, Technical University of Nova Scotia), and Geoff and Alice and their infant daughter (Alison) set off for Halifax in the fall of 1961, full of enthusiasm, in their small, British-built car. Perhaps if they had known at that time just how wide Canada is and just how limited was the automobile, they would not have ventured out! The experience in Halifax was a very good one, however, and they made both personal and professional friends there. Geoff spent two years, most teaching, but also doing a little research, and soon realized that if he wanted to spend his life in this profession he would have to study at the Ph.D. level. He took a leave of absence and entered Lehigh University in the fall of 1964. It was an exciting place to work in the 1960's. Peter Adams had preceded Geoff to Lehigh by one year, and both he and Barb were of great help to the Kulak's. (Worthy of special mention is all the worked Ph.D. Candidacy problems that Peter had amassed—and shared once he knew that he had passed his own Candidacy!)

Geoff worked for John Fisher, and did his Ph.D. research under John's direction. This also started one of those life-long connections that functions on both the professional and personal level. The research, which was in high-strength bolted connections of high-strength steel, started Geoff's research into various aspects of high-strength bolting that has been carried on over the years.

Although there were some interesting job offers upon graduation, both in Canada and in the United States, the Kulak's returned to Halifax, now with two children. In the spring of 1967, Geoff resumed his teaching duties and initiated a more serious research program. However, after three more years in Halifax, the Kulak's realized that they should either buy a boat and settle down to the delightful Maritime life or they should move to a more stimulating intellectual atmosphere. As it happened, there was a opening in structural engineering at the University of Alberta, and Geoff returned to his *alma mater* in 1970. (The vacancy was actually in structural analysis! However, these were the old days, remember, and the Head was able to decide on his own that the position would be filled by someone in steel structures. My, how our lives turn on small details!)

The group in structural engineering in the Department of Civil Engineering at the University of Alberta was a very strong one, and Peter and Geoff, the "steel group", were part of that invigorating atmosphere. Early on in Geoff's career at Alberta, the incumbent Head of Civil Engineering, who had been in the position for fourteen years,

stepped down. Because he had been in the Department only a relatively short time and his shortcomings were still generally unknown to the Department, Geoff was selected as Chairman, a position he occupied for the period 1973–76. He managed to maintain a reasonable level of research during that time, and returned to normal teaching and research activities with vigor in 1976.

Geoff's research interests, although always in the behavior of steel structures and with a strong experimental bias, have had considerable range. His early work in bolted connections continued and then led into similar work in welded connections. This was followed by a period of investigation of local buckling characteristics of structural steel cross-sections, then local buckling of large diameter fabricated steel cylinders, fatigue of welded details, remaining fatigue life of riveted bridge members, fatigue of drill pipe, and so on. A recent, innovative project has been the investigation of steel plate shear walls, a project that is now being investigated together with Laurie Kennedy. In recent years, there have been successful collaborations with those more highly skilled in structural analysis, especially numerical analysis. These collaborations have particularly been with Jim Montgomery, a former staff member, and with Alaa Elwi and Dave Murray.

Transmission of research efforts has always been an important part of the university process. Geoff has been particularly active in code-related work (e.g., Research Council on Structural Connections, Canadian Standards Association codes for steel buildings and for bridges, and international standards) and in publication of material at the student level. He is the co-author of a number of such resources. The undergraduate text, *Limit States Design in Structural Steel*, which he wrote with Peter Adams and Hugh Krentz (later, Mike Gilmor), is in its 5th edition and has been widely used in Canada. The *Guide to Design Criteria for Bolted and Riveted Joints*, written with John Fisher and another graduate student of John's, is the standard reference in this subject area. Geoff has written chapters in other books, and he and a Canadian colleague working in Switzerland, Ian Smith, have written a monograph, *Analysis and Design of Steel Structures for Fatigue: A Primer for Civil Engineers*.

Like others in the Structures Group, Geoff has extensive world-wide contacts with others who work in the area of behavior of steel structures. This has led to periods spent in Brazil (the Canadian International Development Agency project headed by Dave Murray), Lausanne (a six-month study leave spent at the Swiss Federal Institute of Technology) and Norway (Visiting Senior Scientist), as well as lectures or seminars given at various locations throughout the world. These contacts have allowed Geoff and Alice to travel to almost every part of the world.

In 1993, the Carry family of Edmonton, gave the University a large sum of money in honor of their father, who had run a successful medium-size steel fabrication operation in Edmonton for many years. The money was used to establish the C.W. Carry Chair in Steel Structures, and the position was award to Geoff. The endowment is a reflection of the strong steel research group that has worked for many years at the University of Alberta. Most of the income provided by the endowment will be used to fund an entry-level position in steel structures; in this way the group can still be healthy and productive in the future.

Unlike the others being honored at the Symposium, Geoff still has several years left to teach and conduct research before turning over the reins to others. Geoff and Alice are active in the Edmonton community, with strong interests in art and in music, and expect to stay in this area upon retirement.

Professional Career Summary

Geoffrey L. Kulak, Ph.D., P.Eng.

Dr. Kulak is a structural engineer specializing in the behavior and design of structural steel members, connections, and frames. He is a recognized authority on member stability, behavior of welded and bolted connections, and fatigue of fabricated steel members. He has been active in the development of building codes for the design of steel structures and has had a long involvement with the committee that has developed Canada's specification for the design of steel structures. He serves on similar international bodies. The results of his research work are used directly in specifications for the design of steel structures in Canada, the United States, and elsewhere.

EDUCATION

B.Sc. Civil Engineering, University of Alberta, Edmonton, Canada, May 1958
M.S. University of Illinois, Urbana, Illinois, June 1961
Ph.D. Lehigh University, Bethlehem, Pennsylvania, June 1967

EMPLOYMENT HISTORY

Dr. Kulak is Professor of Civil Engineering at the University of Alberta, Edmonton, Canada. In addition to his university duties, he acts as an engineering consultant on problems of an unusual nature to other professional engineers and as an expert in cases involving litigation.

1958 - 1960	Bridge Branch, Department of Highways, Province of Alberta (Design Engineer).
1962	Instructor, Department of Civil Engineering, University of Alberta.
1962 - 1964	Assistant Professor, Department of Civil Engineering, Technical University of Nova Scotia, Halifax.
1964 - 1967	Research Assistant, Department. of Civil Engineering, Lehigh University, Bethlehem, Pennsylvania.
1967 - 1970	Associate Professor, Department of Civil Engineering, Technical University of Nova Scotia.
1970 - Present	Department of Civil Engineering, University of Alberta. Professor since 1973, Chairman 1973-76.

PROFESSIONAL AND TECHNICAL ASSOCIATIONS

Dr. Kulak is a registered Professional Engineer in the Province of Alberta. He is a Member of the Canadian Society for Civil Engineering, a Fellow of the American Society of Civil Engineers, and a member of the International Association for Bridge and Structural Engineering. He is active in a number of technical committees related to the behavior of steel structures. These include the Research Council on Structural Connections (member of the Council and member of the Specifications Committee), Canadian Standards Association Committee S16 – Steel Structures for Buildings (Vice-Chairman), and Canadian Standards Association Committee S6 – Canadian Highway Bridge Design Code.

ACADEMIC HONOURS AND AWARDS

- Invited Visitor, Pontificia Universidade Catholic, July 1983 – Rio de Janeiro, Brazil. As part of a Canadian International Development Agency project, Dr. Kulak spent one month at this university giving lectures and seminars to graduate students and to practicing engineers.
- Invited Overseas Visitor, South African Institute of Steel Construction, 1984 and Invited Professor, Dept. of Civil Engineering, University of Pretoria and Invited Professor, Dept. of Civil Engineering, University of Stellenbosch, Republic of South Africa in July, 1984. In August 1991, Dr. Kulak was one of the keynote speakers at the 75th Anniversary Conference of the South African Institute of Steel Construction.
- Invited Professor, Institute Construction Metallique, Ecole Polytechnique Federale de Lausanne, Switzerland – Jan. 1984 to June 1984.
- Moisseff Medal of the American Society of Civil Engineers, 1985, (with former graduate student, J.L. Dawe) – this is one of the senior awards of the Society and is given annually for the best paper on a topic in structural engineering.
- Special Merit Supplement, Natural Sciences and Engineering Research Council, 1988. (Special awards above the normal operating grant were given to selected researchers.)
- Visiting Senior Scientist, Institute for Steel Construction, Norwegian Institute of Technology, Trondheim, Norway, July 1, 1992 to December 31, 1992

PUBLICATIONS

Dr. Kulak is the author or co-author of over seventy technical publications and five books. Many of the technical publications concern the strength and behavior of high-strength bolts and bolted joints. He is a co-author of the principal North American source for technical information on bolting in structural steelwork: "Guide to Design Criteria for Bolted and Riveted Joints," by G.L. Kulak, J.W. Fisher, and J.H.A. Struik, Second Edition, John Wiley and Sons, New York, 1987. He is also a co-author of the

undergraduate steel design textbook in common use in Canada: "Limit States Design in Structural Steel," by G.L. Kulak, P.F. Adams, and M.I. Gilmor, Fourth Edition, Canadian Institute of Steel Construction, 1990. The following is a selected list of publications.

Books or Chapters in Books

Kulak, G.L., Fisher, J.W., and Struik, J.H.A., 1987. Guide to Design Criteria for Bolted and Riveted Joints Second Edition, John Wiley and Sons, Inc., New York.

Kulak, G.L., Adams, P.F., and Gilmor, M.I., 1990. Limit States Design in Structural Steel, Fourth Edition, Canadian Institute of Steel Construction, Toronto.

Kulak, G.L., 1991. Structures Subjected to Repeated Loading – Stability and Strength, Chapter 9 (Unstiffened Steel Plate Shear Walls), Elsevier.

Papers

Fisher, J.W., Galambos, T.V., Kulak, G.L., and Ravindra, M.K., 1978. Load and Resistance Factor Design Criteria for Connectors, Journal of the Structural Division, ASCE, Vol. 104, No. ST9, September.

Dawe, J.L. and Kulak, G.L., 1984. Plate Instability of W Shapes, Journal of the Structural Division, ASCE, Vol. 110, No. 6, June.

Dawe, J.L. and Kulak, G.L., 1984. Local Buckling of W Shape Columns and Beams, Journal of the Structural Division, ASCE, Vol. 110, No. 6, June.

Kulak, G.L. and Fisher, J.W., 1985. Behavior of Slip-Resistant Joints, Periodica, International Association for Bridge and Structural Engineering, November.

Kulak, G.L., 1986. Behaviour of Unstiffened Steel Plate Shear Walls, Proceedings, Pacific Structural Steel Conference, Auckland, New Zealand, August.

McKenzie, K.A. and Kulak, G.L., 1987. Mall Coaster Inquiry Report, Report to the Government of Alberta, August, 235 pp.

Kulak, G.L. and Baker, K.A., 1989. Fatigue Strength of a Groove Weld on Steel Backing, Journal of Constructional Steel Research, Vol. 12.

Obaia, K.H., Elwi, A.E., and Kulak, G.L., 1992. Ultimate Shear Strength of Large Diameter Fabricated Steel Tubes, J. Construct. Steel Research, Vol. 22, pages 115-132.

Mohareb, M., Alexander, S.D.B., Kulak, G.L., and Murray, D.W., 1993. Laboratory Testing of Line Pipe to Determine Deformational Behavior, Proceedings of the 12th Inter. Conf. on OMAE, Vol. V-Pipeline Technology, ASME, pp. 109-114.

Kulak, G.L. and Smith, I.F.C., 1993. Analysis and Design of Fabricated Steel Structures: A Primer for Civil Engineers, Structural Engineering Report No. 190, Department of Civil Engineering, University of Alberta, July.

GRADUATE STUDENTS SUPERVISED BY G.L. KULAK

Dawe, J.L.	M.Sc.	1972
Perlynn, M.J.	M.Sc.	1974
Nash, D.S.	M.Sc.	1976
Bardell, G.	M.Sc.	1978
Comeau, M.P.	M.Sc.	1979
Dawe, J.L.	Ph.D.	1980
Ogle, R.B	M.Sc.	1981
Baker, K.A.	M.Sc.	1982
Brah, K.	M.Eng.	1982
Stephens, M.J.	M.Sc.	1982
Thorburn, L.J.	M.Sc.	1982
Filipiak, R.M.	M.Eng.	1983
Bailey, R.W.	M.Sc.	1984
Timler, P.A.	M.Sc.	1984
Dobslaw, W.	M.Eng.	1985
Green, D.L.	M.Sc.	1987
Tromposch, E.W.	M.Sc.	1987
Hust, D.	M.Eng.	1988
Grondin, G.Y.	Ph.D.	1991
Obaia, K.H.	Ph.D.	1991
Chen, Qishi	Ph.D.	1993
Wu, Y.	M.Sc.	1993
Undershute, S.	M.Sc.	1994

James G. MacGregor

A Personal History

James Grierson MacGregor was born in Vegreville, Alberta on February 14, 1934, the oldest child of Jim and Marjorie MacGregor. He attended Sunalta School and Central Collegiate Institute, both in Calgary. Jim enrolled in Civil Engineering at the University of Alberta in September, 1952. In May 1956 he graduated with Distinction and received the APEGGA Gold Medal in Civil Engineering.

Following graduation Jim worked in the Calgary office of Structural Engineering Services, the forerunner of Lamb McManus Associates. On September 1, 1956, Jim married Barb Dawson who had majored in Psych and bridge. Their honeymoon was spent traveling via the Lake of the Woods to the University of Illinois where Jim enrolled in a Master's program on September 7, 1956. Apart from courses the first year was spent working as Joe Warwaruk's assistant in the structures lab.

Barb and Jim's first son, Bob, was born in Champaign-Urbana on November 10, 1957. Shortly after, the family bought its first car, a 1950 DeSoto Sedan that "had belonged to the wife of a retired university professor". Jim completed the work for a Master of Science in May 1958 and the family returned to Edmonton for the summer driving the DeSoto. The summer was spent working for Rensaa and Minsos, Architects and Engineers.

The MacGregor family returned to Urbana in the fall of 1958 and Jim started work on his doctor's thesis. About this time, Jim's advisor, Chet Siess offered Jim a course credit if Jim would draft answers for Chet's ACI Code correspondence. This was a great learning experience. Jim seldom hit the nail on the head in the first draft and did a great amount of library research and number crunching to prepare the needed answers. At the end of the semester Jim got the course credit but continued in this role for the rest of his stay at Illinois.

Barb and Jim's daughter, Joan was born in Champaign-Urbana on August 27, 1959. Jim passed his Ph.D. thesis oral in July, 1960 and the family left Urbana the same day to return to Edmonton where Jim joined the staff of the Civil Engineering Department on September 1, along with Dave Murray, Joe Warwaruk and Pete Adams. The first year the MacGregor family lived next door to the Murrays in the New Staff

Member's Housing on the NW corner of 112 Street and 87 Avenue (better known as the Rabbit Hutches). The family moved into a new house in Rio Terrace the next summer. Their third child and first Canadian, Dave, was born on March 24, 1962.

In 1961 Jim joined the newly formed ACI-ASCE Committee on Reinforced Concrete Columns along with his friends Jack Breen and Ed Pfrang. Other members of the committee included Boris Bresler and Ignacio Martin. This started Jim's lifelong involvement with ACI.

Jim worked summers and other times with Whittaker Carsewell Consulting Engineers from 1961 to 1967. Part of this time was spent designing parts of the Thaketa Bridge in Rangoon, Burma, a steel bascule bridge with prestressed concrete approach spans. Jim joined the Kinsmen Club of Edmonton in 1962. During the construction of the Kinsmen Field House in 1967-68, Jim chaired the Field House Committee. In 1967, Pete Adams and Jim formed Adams and MacGregor Consulting Engineers Ltd.

During the 1960's and 1970's the family spent time each summer camping, traveling, fishing and hiking. Major trips were made to the Tonquin Valley on two occasions, to Mt. Assiniboine with Bob when he was 8, to Lake O'Hara and to Egypt Lake.

In 1967 Jim was named to the Canadian Technical Committee on Reinforced Concrete Design. He became Chairman of the committee in 1976 and was in that position when the 1977 and 1984 codes were published. In 1968 he was named as Canadian delegate to the European Concrete Committee (CEB) and made the first of a number of trips to Europe. From 1978 to 1980 Jim was Vice-President of CEB.

On June 21, 1969 the MacGregor family flew to London, England where they took delivery of an English milk truck fitted out as a camper (promptly named "Gus", short for "Galapogus" as in giant land turtles). This was their home for the next five weeks while they made their way to Stuttgart where Jim spent six months on sabbatical leave at the Otto Graf Institute working with Rene Walther under the direction of Professor Leonhardt. Joan and Dave attended the local German schools while Bob took junior high school in the American Army school. Every weekend in the fall was spent exploring southern Germany and Switzerland. In late February 1970 the family set out again, this time for southern France, Pisa, Florence, Venice, through Yugoslavia to Greece, on to Turkey and eventually back to Braunschweig, Germany where Jim spent two months with Professor Kordina. In July 1970 the MacGregor family returned to Halifax on the Nieuw Amsterdam and set out to cross Canada to Edmonton. This year had a profound effect on the entire family which developed a great closeness and a great appreciation for other ways of life.

In 1972 Jim, Jack Breen and Ed Pfrang received the Wason Medal for Most Meritorious Paper and the Raymond C. Reese Medal for structural research from the American Concrete Institute for a co-authored paper on slender columns. This was the first of several awards Jim shared with his colleagues or students.

1978-79 was spent on sabbatical leave at the University of Texas where Barb, Joan, Dave and Jim learned the finer points of Texas barbecue. Jim worked on the first draft of his textbook (finally published in 1987), Barb toured the United States as a

trainer for the Association of Junior Leagues and our son Dave played men's league hockey at a rink in a local shopping mall. In June we traveled to Gaithersburg, Maryland where Jim worked for a month with Bruce Ellingwood, Ted Galambos and Allin Cornell at the National Bureau of Standards. During this time the load factors used in the AISC LRFD Code were developed.

In May of 1979 Barb and Jim's son Bob became the first, third-generation graduate in engineering from the University of Alberta. Bob worked for Lamb McManus Associates in Edmonton until 1984. He received a Master of Engineering in 1984 from the University of Alberta and a Ph.D. from the University of Texas in 1989.

In 1981, Dave Murray, Geoff Kulak and Jim formed MKM Engineering Consultants.

Jim was elected a Fellow of the Academy of Sciences of the Royal Society of Canada in 1982 and was promoted to University Professor in 1985. In 1989 he was elected as a Fellow of the Canadian Academy of Engineers. He is also a Fellow of ACI, ASCE and CSCE.

Kyle, the first of Barb and Jim's seven grandchildren was born to their daughter Joan in 1984. He was followed by a brother Neal, Bob's children Lisanne, Andy and Sean, and Dave's children Tyler and, earlier this year, Amie Mae Jean.

In 1984 Jim was the Eminent Foreign Lecturer for the Institution of Engineers of Australia. They saw him coming. The schedule included 48 lectures, seminars or meetings in a six week visit to eight cities. Barb joined Jim three weeks into the trip in time for a train trip from Perth to Adelaide. After a week in New Zealand and a week in Edmonton, Barb and Jim flew to Rio de Janiero for a memorable month in Brazil centered on a visit to PUC-RJ. From 1987 to 1990, Jim served as Chairman of the Department of Civil Engineering. This was a stressful time. The downsizing, which the University instituted during Dave Murray's term as Chairman, continued, forcing Jim to discharge several long term employees of the Department. In 1990 Jim gave the Gordin J. Kaplin Lecture representing the sciences.

Late 1991 to 1993 were eventful years, beginning sadly with the death of Bob when Joan's house was destroyed by fire. On a happier note, Jim became President of the American Concrete Institute in March 1992. This involved visits to chapters in nine countries and hosting conventions in San Juan, Puerto Rico and Vancouver. Jim was diagnosed with Parkinson's Disease in 1992 which led to his decision to take early retirement as of August 31, 1993, ending 33 years of service on the faculty. In 1993 his son Dave graduated from Technical University of Nova Scotia with a Master of Architecture. Construction started on Barb and Jim's new home at Halfmoon Bay, B. C. in late summer 1993. The contractors for the house were their daughter Joan and her husband Kit Roberts a situation which proved ideal when building at a long distance.

After 28 years in their Windsor Park home, Barb and Jim made the move to the coast, where they have family and friends, in early May, 1994. They look forward to gardening, the next editions of Jim's book and exploring their new surroundings. With Mariann (Bob's wife) and Dave both living in Edmonton there will be many trips back and forth.

Professional Career Summary

J.G. MacGregor, Ph.D., P.Eng., FRSC, FCAE

PERSONAL INFORMATION

Born:	Vegreville, Alberta, February 14, 1934
Citizenship:	Canadian
Married:	September 1, 1956, 3 children, one deceased
Address:	R.R. 1 Curran Site C36 Halfmoon Bay, B.C. V0N 1Y0

EDUCATION

B.Sc.	Civil Engineering (with Distinction), University of Alberta, Edmonton, May 1956
M.S.	Civil Engineering, University of Illinois, Urbana, October 1958
Ph.D.	Civil Engineering, University of Illinois, Urbana, October 1960

EMPLOYMENT

1956 (Summer)	Structural Engineering Services, Calgary, Engineer
1958 (Summer)	Rensaa and Minsos Limited, Edmonton, Engineer
1960 - 1963	University of Alberta, Assistant Professor
1963 - 1968	University of Alberta, Associate Professor
1961 - 1967	Whittaker Carsewell Consulting Engineers Limited, Engineer
1967 - 1981	Adams and MacGregor Consulting Engineers, Limited, Principal
1968 - 1985	University of Alberta, Professor
1981 - Present	MKM Engineering Consultants, Principal
1985 - 1993	University of Alberta, University Professor
1987 - 1990	University of Alberta, Chairman, Department of Civil Engineering
1993 - Present	University Professor Emeritus

AWARDS

Association of Professional Engineers of Alberta, Gold Medal in Civil Engineering, 1956.

American Society of Civil Engineers, State of the Art of Civil Engineering Award, 1968. For the paper entitled "Review of Concrete Beams Failing in Shear", jointly with B. Bresler.

American Concrete Institute, Wason Medal for Most Meritorious Paper, 1972. "For the most meritorious paper presented to the Institute", jointly with J.E. Breen and E.O. Pfrang. For the paper "Design of Slender Concrete Columns".

American Concrete Institute, Raymond C. Reese Medal for Structural Research, 1972. "To recognize outstanding contributions to the application of structural engineering research", jointly with J.E. Breen and E.O. Pfrang. For the paper "Design of Slender Concrete Columns"

Province of Alberta, Alberta Achievement Award, 1972. "For Excellence in Structural Engineering."

American Concrete Institute, Elected as a Fellow of the Institute in the first election of Fellows, 1973.

Government of Canada, Dinner with Queen Elizabeth II for Young Canadians in the Arts and Sciences, 1975.

American Concrete Institute, Delmar L. Bloem Distinguished Service Award, 1974.

American Society of Civil Engineers, State of the Art of Civil Engineering Award, 1974. For the report entitled "The Shear Strength of Reinforced Concrete Members", prepared by the ASCE-ACI Committee on Shear and Diagonal Tension, chaired by J.G. MacGregor.

American Society of Civil Engineers, Raymond C. Reese Research Prize, 1976. "To recognize outstanding contributions to the application of structural engineering research." For the report entitled "The Shear Strength of Reinforced Concrete Members - Slabs", jointly with the ASCE-ACI Committee on Shear and Diagonal Tension.

American Society of Civil Engineers, Elected Fellow of the Society, 1978.

Canadian Society for Civil Engineering, Certificate of Appreciation, 1978. "For his national lecture tour 1976."

American Society of Civil Engineers, Raymond C. Reese Research Prize, 1979. "To recognize outstanding contributions to the application of structural engineering research." For the paper entitled "Stability Analysis and Design of Concrete Frames", jointly with S.E. Hage.

American Society of Civil Engineers, Can-Am Civil Engineering Amity Award, 1979. "For active participation in technical committees of Canadian and American Engineering Societies and his contributions to the engineering profession through his technical papers."

Royal Society of Canada, Elected as a Fellow of the Academy of Sciences, May 1982.

Canadian Society for Civil Engineering, Elected as a Fellow of the Society, June 1983.

American Society of Civil Engineers, Norman Medal, 1983. "For a paper judged worthy of special commendation for its merit as a contribution to engineering science", jointly with B. Ellingwood, C.A. Cornell and T.V. Galambos.

Reinforced Concrete Research Council, A.J. Boase Medal, 1985. "For his noteworthy research on reinforced concrete and his sustained and effective contributions to the preparation of building codes in both the U.S. and Canada."

American Concrete Institute, Joe W. Kelly Award, 1986. "For research and teaching of student and practitioner."

Association of Professional Engineers Geologists and Geophysicists of Alberta, Centennial Award, 1986. "...who through research and consulting has attained unusual distinction as a recognized authority on design and analysis of concrete structures."

American Concrete Institute, Raymond C. Reese Research Medal, jointly with D.M. Rogowsky, 1987. "To recognize outstanding contributions to the application of structural engineering research". For the paper "Design of Reinforced Concrete Deep Beams".

American Concrete Institute, First Annual Phil M. Ferguson Lectureship, 1988.

Canadian Academy of Engineering, Elected a Fellow of the Academy, 1989.

University of Alberta, Gordin J. Kaplin Research Prize, 1990.

Canadian Standards Association, Award of Merit, 1991.

VISITING LECTURESHIPS

Canadian Society for Civil Engineering, Second Annual Travelling Lectureship, "Safety and Limit States Design for Reinforced Concrete". Vancouver, Calgary, Edmonton, Regina, Saskatoon, Winnipeg, Toronto, Ottawa, Niagara Falls, Montreal, Quebec, Fredericton, Halifax, 1975- 1977.

Peoples Republic of China, Academy of Building Research, "Structural Safety", Beijing and Shanghai, 1981.

Instituto Mexicano del Cemento y del Concreto, "Design of Concrete Floor Slabs", Chihuahua and Guadalajara, 1981.

Institution of Engineers, Australia, Civil College Eminent Overseas Speaker, Sydney, Newcastle, Brisbane, Canberra, Cooma, Perth, Bunbury, Adelaide, Port Pirie, Melbourne and Hobart, Australia and Christchurch, New Zealand, May- July 1984.

Portland Cement Institute, Republic of South Africa, Johannesburg, Cape Town and Durban, July 1986.

VISITING PROFESSORSHIPS AND VISITING RESEARCHSHIPS

University of British Columbia - 1975

University of Texas at Austin - 1976, 1978-79

National Bureau of Standards, Gaithersburg, Maryland, - 1979

University of Witswatersrand, Johannesburg, - 1986

PROFESSIONAL AND TECHNICAL ASSOCIATIONS AND COMMITTEES

Association of Professional Engineers, Geologists and Geophysicists of Alberta

- Professional Engineer
- Public Relations Committee
- Council
- Discipline Committee

Canadian Society for Civil Engineering

Canadian Standards Association

- Steering Committee on Structures (Design)
- Task Committee on Limit States Design
- Technical Committee on Reinforced Concrete Design
  - Chairman - 1976 - 1989

National Building Code of Canada

- Joint Committee on Reinforced Concrete Design
- Standing Committee on Structural Design
  - Chairman - 1989 - Present

American Concrete Institute

- Fellow
- Board of Direction
  - Board Member
  - Vice-President
  - President
- Committee 318 - Standard Building Code
  - Member of Steering Committee
  - Chairman of Subcommittee E, Shear
- Committee 442 - Lateral Forces on Tall Buildings
- Technical Activities Committee
- Committee 426 - "Shear and Diagonal Tension"
  - Chairman - 1970 - 1975
- Committee 441 - "Reinforced Concrete Columns"

American Society of Civil Engineers

- Fellow
- Committee on Masonry and Reinforced Concrete

- Committee on ASCE7, "Design Loads for Buildings"

Comité Euro-International du Béton, Lausanne

- Expert Member
- Canadian Delegate
- Vice-President - 1978 - 1980

International Association for Bridge and Structural Engineering, Zurich

- Commission III - "Concrete, Reinforced and Prestressed Concrete"
  Vice-Chairman

COMMUNITY ASSOCIATIONS AND COMMITTEES

Kinsmen Club of Edmonton

- Kinsmen Field House Building Committee
  Chairman - 1967 - 1968 (during construction)
- Director - 1967

UNIVERSITY COMMITTEES

Academic Development Committee

- Member - 1988 - 1989

General Faculties Council

- Member (of first elected council) - 1966 - 1969
- GFC Executive - 1966 - 1969

Campus Development Committee

- Member - 1975 - 1978

Priorities and Planning Committee

- Member - 1986 - 1989

Civil Engineering Department

- Chairman - 1987 - 1990

Canadian Council of Professional Engineers

- Member of Accreditation Team - University of Manitoba
- Member of Accreditation Team - University of British Columbia

External Examiner on Ph.D. Committees

- University of Canterbury, Christchurch, N.Z., 1970
- University of Waterloo, Waterloo, Ontario, 1973
- Indian Institute of Technology, Delhi, 1974
- University of Canterbury, Christchurch, N.Z., 1974
- University of Toronto, Ontario, 1975

- University of Waterloo, Waterloo, Ontario, 1976
- University of New South Wales, Kensington, Australia, 1982
- University of Auckland, Auckland, N.Z., 1983
- University of Canterbury, Christchurch, N.Z., 1985
- University of Witswatersrand, Johannesburg, R.S.A., 1988, 1991

PUBLICATIONS

Refereed Journals or Refereed Symposium Volumes

- 95 papers

Symposia and Monographs Edited

- 6 Chapters or Volumes edited.

Books

MacGregor, J.G., Reinforced Concrete: Mechanics and Design, Prentice-Hall, November 1987, 784 pp. Second Edition, 1992, 848 pp. Adoptions include University of California at Berkeley, Georgia Institute of Technology, University of Illinois at Urbana, Purdue University, Michigan State University, Ohio State University, Texas A and M University.

David W. Murray

A Personal History

David William Murray was born in Calgary in 1930, the second son of Elmer E. and Elsie Murray. The public schools that he attended were Mount Royal School, Sunalta Junior High School and Central High School. When he matriculated in 1948 his family moved to Edmonton so that he could attend the University of Alberta. He transferred to the civil engineering program after completing a year in the Science Faculty. In addition to his studies he was active in the University Symphony Orchestra, in which he played the flute, and was the concert soloist in his final year. Upon graduation he received the APEGGA Gold Medal in Civil Engineering and went to Imperial College in London, England, as an Athlone Fellow. He completed the requirements for a Diploma of Imperial College in Water Power Engineering in 1953, and for a Master of Science in Civil Engineering from the University of London in 1954.

Upon returning from England, Dave worked in the Edmonton area for the firm of Lamb, McManus Consulting Engineers for a year and a half, and then went to Montreal to work in the design office of Montreal Engineering Company Limited. He returned to Edmonton in August 1956 to marry Pearl Bidulock from Hairy Hill, Alberta, and she accompanied him back to Montreal where she taught school in the town of Mount Royal. In August of 1957 they moved to Winnipeg, Manitoba, where Dave took a cut in pay in order to join the Academic Staff of the University of Manitoba as an Assistant Professor at a salary of $4500 per year. During their three year tenure in Winnipeg their two daughters, Natalie and Charlene, were born, and Dave worked as a structural designer with the office of Green, Blankstein and Russell, Architects and Engineers, in the summer periods.

In 1960, Dave's father became ill and had a major operation. So during their annual visit to their families in Alberta, Dave dropped in for a visit with Dr. S. Sinclair, then Chairman of the Civil Engineering Department, and inquired if there would be a possibility to join the faculty of the University of Alberta. He had an interview with Dean Govier the following morning and accepted the offer of a position that afternoon. Dave and Pearl moved into the staff apartments on 87 Ave. in August of 1960 (next door to the MacGregors) and purchased their home in Windsor Park the following year. Their boys, Douglas and Donald, were born in Edmonton.

With the emphasis on engineering research brought about by the space race, and the restructuring of engineering education in North America, it soon became apparent that Dave's engineering credentials would not be adequate to function effectively in a major university, such as the University of Alberta aspired to become. Consequently, Dave made arrangements to attend the 10 week NSF Summer Institutes to upgrade college teachers in applied mechanics, held at the University of Colorado, in Boulder, Colorado, during the summers of 1963 and 1964. Travel to and from Colorado for these courses occurred in the form of family tenting expeditions. Encouraged by a successful experience in 1963, Dave applied for, and was accepted for, graduate studies in structural engineering at the University of California, Berkeley, for the fall of 1964. So, at 34 years of age and with his wife and four small children in tow, he left for California and the Murrays began their Ph.D. research program housed in a two-story two-bedroom apartment in the University Village at Albany, across the bay from San Francisco.

During the period of residence in California, when each school year was over in late June, Pearl used to pack the children in the station wagon and set off alone on the long trek northward, back to the family homestead in Hairy Hill, in order to spend the summer months in a less confined environment. Dave joined them in late August for a break of a week or two.

Upon completion of Dave's Ph.D. in 1967, the Murrays returned to their house in Windsor Park. Pearl now had a chance to upgrade her credentials by returning to University (part-time, in view of her responsibilities at home) and completed in 1970 the B.Ed. degree that she had started approximately 20 years earlier, with a major in art. The travel bug was rather deeply ingrained by this time and the family returned to California in 1969 for holidays. But Don, the youngest, developed appendicitis on the trip. He was operated on in the Children's Hospital in Oakland the day they arrived in California. This deprived the rest of the children of a visit to Disneyland. They never forgave him! The family tried again, and undertook a cross-country trip to New York and Halifax in 1970. While Dave attended a two week summer course in Cornell, Pearl and the children went sight-seeing to New York City, Washington and New Jersey. In 1972, Dave spent 6 months of his first sabbatical at the University of Wales in Swansea. The boys went to a Welsh elementary school while the girls studied correspondence courses at home in a cottage by the sea on the Gower Peninsula. During this period the family undertook a six week European tour that extended as far as Turkey and up through the Balkans to the southern edge of the Soviet Union, where Pearl found some of her father's family. This trip left a lasting impression on the entire family.

Upon returning to Edmonton after the Welsh sabbatical, life settled into a relatively peaceful routine. This was disrupted for approximately a five year period, beginning in 1974, during which all available time was consumed by a research project, shared with Dr. MacGregor and Dr. Simmonds, on the over-pressure behavior of Canadian prestressed concrete containment structures. The associated activity continued until the welcome break of another sabbatical in 1979-80, of which the first six months were spent in Brazil at the Pontificia Universidade Católica do Rio de Janeiro (PUC-Rio). The boys attended the Escola Americana while Charlene and Pearl studied Portuguese at the Instituto Brasil-Estados Unidos. Natalie remained in Edmonton to work. The second

half of the sabbatical was spent at the Institüt für Baustatik of the University of Stuttgart with Dr. Ekkehard Ramm. Now Natalie and the boys remained at home, while Charlene studied German at the Goethe Institute in Mürnau.

In 1982, Dave and Dr. Eisenstein became codirectors of a CIDA program to upgrade the graduate education and research programs in civil engineering at PUC-Rio. The application to CIDA for this project arose out of the previous Brazilian sabbatical. A close relationship with PUC-Rio has continued to the present day, with one month visits occurring in 1982 and 1984, and five month visits in 1988, 1991 and 1993. Dave was awarded the title of Professor Honorário of PUC-Rio in 1991.

A very demanding period occurred from 1982 to 1987, during Dave's five year tenure as Department Chairman. At the end of this period Dave and Pearl spent the first half of his administrative leave at the University of California, Berkeley, and the remaining half at PUC-Rio. Dave's early retirement was effective on December 31, 1991 after 33.5 years of pensionable service. This was followed immediately by a two-year half-time appointment and his designation as an Emeritus Professor. The part-time appointment expired on December 31, 1993, but Dave continues an active research program and is presently co-supervising three graduate students and a post-doctoral fellow.

Dave and Pearl have no plans to leave the Edmonton area, where Natalie and her family, and Douglas, have their homes. Charlene and her family live in Silver Valley, Alberta, and Donald has recently obtained a position with an engineering firm in Vancouver. Pearl intends to pursue more vigorously her activities as an artist, grandchildren permitting, and Dave may do a little consulting along with his research.

Professional Career Summary

David W. Murray, Ph.D., P.Eng.

Dr. Murray is a structural engineer who has specialized in the analysis of structural systems for the purpose of predicting their realistic behavior. The basic methodology he has used for this purpose has been the stiffness method of structural analysis combined with finite element formulations, including the incorporation of nonlinear material and nonlinear geometric effects. Among the models he has assisted in developing are those for the prediction of the behavior of thin-walled steel structures, steel frames, reinforced concrete slabs, prestressed concrete shells and containment structures, reinforced concrete beams and panels, and buried steel pipelines. Earlier in his career he also developed solutions for thermal analysis of thawing soils and the freeze-thaw cracking behavior of pavements.

EDUCATION

1952	B.Sc.	(Civil Engineering) University of Alberta, Edmonton
1953	D.I.C.	(Civil Engineering) Imperial College, London
1954	M.Sc.	(Engineering) University of London, London
1967	Ph.D.	(Structural Engineering) University of California, Berkeley

ACADEMIC HONOURS AND AWARDS

1948	Viscount Bennett Matriculation Scholarship
1949	First Class Standing Prize
1950	First Class Standing Prize
1951	University of Alberta Honor Prize
	Viscount Bennett Undergraduate Scholarship
1952	Association of Professional Engineers Gold Medal
	The Athlone Fellowship
1991	Honorary Professor, Pontificia Universidade Católica do Rio de Janeiro.

ACADEMIC EXPERIENCE

Professor Emeritus	Civil Engineering	Univ. of Alberta	1992-Present
Chairman	Civil Engineering	Univ. of Alberta	1982-87
Professor	Civil Engineering	Univ. of Alberta	1970-1991
Assoc. Professor	Civil Engineering	Univ. of Alberta	1961-69
Teaching Fellow	Civil Engineering	Univ. of California	1964-67
Assoc. Professor	Civil Engineering	Univ. of Alberta	1960-61
Asst. Professor	Civil Engineering	Univ. of Manitoba	1957-60

VISITING PROFESSORSHIPS

University	Location	Period
University of Stuttgart	Stuttgart, Germany	1/2/80-31/7/80
Pontificia Universidade Católica do Rio de Janeiro	Rio de Janeiro, Brazil	1. 01/06/79-31/11/79 2. 18/02/88-18/06/88 3. 01/01/91-31/05/91 4. 08/02/93-31/05/93

INDUSTRIAL EXPERIENCE

1954-56: Design Engineer: T. Lamb, R.N. McManus and Associates, Consulting Engineers, Edmonton, Alberta.

1956-57: Structural Design Group Leader: Montreal Engineering Company Ltd., Montreal, Quebec.

1957-60: Structural Engineer: Green, Blankstein, Russel and Associates, Architects and Consulting Engineers, Winnipeg, Manitoba. (Part-time)

1960-94 Numerous individual consulting assignments.

GUEST LECTURES IN FOREIGN COUNTRIES

Guest lectures on research into the behavior of prestressed concrete secondary containment structures have been presented as follows.

(a) Pontificia Universidade Católica do Rio de Janeiro, Aug.-Sept. 1979.
(b) The Second South American Conference on Numerical Methods in Civil Engineering, Porto Allegre, Brasil, Dec. 1979.
(c) University of Stuttgart, Stuttgart, Germany, May 1980.
(d) University of Darmstadt, Darmstadt, Germany, June, 1980.
(e) University of the Ruhr, Bochum, Germany, June, 1980.
(f) University of Braunschweig, Braunschweig, Germany, June 1980.
(g) Conference on Design, Testing and Construction of Nuclear Containment Structures, Pontificia Universidade Catolica do Rio de Janeiro, Brazil, June 24 and 25, 1982.
  - Typical Nuclear Containment Systems
  - Structural Behavior of Elements of Prestressed Nuclear Containment Structures
  - Design and Testing of a Model Nuclear Containment Structure
  - Analysis to Failure of Prestressed Nuclear Containment Structures

Guest lectures on research into the behavior of structures in general have been presented as follows.

(h) Petrobras, Rio de Janeiro, June 15, 1982.
  - Buckling of Steel Shells

- (also presented at Instituto Technológico Aernota'utica, São Jose dos Campos, Brazil, June, 1982; and at COPPE, Rio de Janeiro, June 21, 1982)

(i) Instituto Pesquisas Technológicas do Estado de São Paulo, Brazil, June 30, 1982.
- Finite Element Techniques for Nonlinear Analysis of Concrete Structures
  (also presented at COPPE, Rio de Janeiro, June 21, 1982)

(j) Pontificia Universidade Católica do Rio de Janeiro, Brazil, June 22, 1982.
- Case Histories to Illustrate Some Unusual Structural Problems

(k) Pontificia Universidade Católica do Rio de Janeiro, Brazil, July 1984.
- Ponding Behaviour and a Warehouse Roof Failure
- Hybrid Finite Elements for Concrete Slab Design
- Field Measurements and Analytical Studies into Overstressing of a Strip Mine Dragline Base
- Some Aspects of Science Policy in Canada

(l) University of California, Berkeley, November 1987
- Nonlinear Analysis of Reinforced Concrete Beams by the Finite Element Method
  (Also presented at Pontificia Universidade Católica do Rio de Janeiro, Brazil, April, 1988, and at COPPE, the Federal University of Brazil, Rio de Janeiro, June 15, 1988.)

(m) Pontificia Universidade Católica do Rio de Janeiro, Brazil, April, 1991
- Introduction to Nonlinear Solution Techniques for Softening Structures
- Reduction Methods for Solution of Finite Element Formulations of Nonlinear Structural Behavior

(n) Pontificia Universidade Católica do Rio de Janeiro, Brazil, April, 1993
- A Model for Interactive Soil-Structure Pipeline Settlement Analysis
- Observations on Behavior of Line Pipe for the Oil Industry

PUBLICATIONS IN REFEREED JOURNALS AND CONFERENCE PROCEEDINGS

Dr. Murray is author or coauthor of over 60 publications in refereed journals and proceedings. A selection of some of these is as follows.

Murray, D.W. and Wilson, E.L., 1968. An Approximate Nonlinear Analysis of Thin Plates, Second Conference of Matrix Methods in Structural Mechanics, Wright-Patterson Air Force Base, Ohio, October, pp. 1207-1230.

Murray, D.W. and Wilson, E.L., 1969. Large Deflection Plate Analysis by Finite Element, Journal of the Engineering Mechanics Division, ASCE, Vol. 95, EM1, February, pp. 143-165.

Murray, D.W. and Wilson, E.L., 1969. Finite-Element Post-Buckling Analysis of Thin Elastic Plates, A.I.A.A. Journal, October, pp. 1915-1920.

Hwang, C.T., Morgenstern, N.R. and Murray, D.W., 1971. On Solutions of Plane Strain Consolidation Problems by Finite Element Methods, Canadian Geotechnical Journal, Vol. 8, No. 1, February, pp. 109-118.

Rajasekaran, S. and Murray, D.W., 1971. Inelastic Buckling of Thin-Walled Members, Proceedings of the First Specialty Conference in Cold-Formed Steel Structures, University of Missouri-Rolla, August 19-20, pp. 43-51.

Hwang, C.T., Murray, D.W. and Brooker, E.W., 1972. A Thermal Analysis for Structures on Permafrost, Canadian Geotechnical Journal, Vol. 9, No. 1, February, pp. 33-46.

Christison, J.T., Murray, D.W. and Anderson, K.O., 1972. Stress Prediction and Low Temperature Fracture Susceptibility of Asphaltic Concrete Pavements, Proceedings of the American Association of Asphalt Paving Technologists, Cleveland, Ohio, February, pp. 494-523.

Scanlon, A. and Murray, D.W., 1972. An Analysis to Determine the Effects of Cracking in Reinforced Concrete Slabs, Specialty Conference on the Finite Element Method in Civil Engineering, McGill University, June, pp. 841-867.

Rajasekaran, S. and Murray, D.W., 1973. Finite Element Solution of Inelastic Beam Equations, Journal of the Structural Division, ASCE, Vol. 99, No. ST6, June, pp. 1025-1041.

Rajasekaran, S. and Murray, D.W., 1973. Coupled Local Buckling in Wide-Flange Beam-Columns, The Journal of the Structural Division, ASCE, Vol. 99, No. ST6, June, pp. 1003-1023.

Rajasekaran, S. and Murray, D.W., 1973. Incremental Finite Element Matrices, Journal of the Structural Division, ASCE, Vol. 99, ST12, December, pp. 2423-2438.

Rajasekaran, S. and Murray, D.W., 1974. Finite Element Large Deflection Analysis of Thin-Walled Beams of Open Section, Proceedings of the International Conference on Finite Element Methods in Engineering, University of New South Wales, Kensington, Australia, August 28- 30, pp. 435-454.

Scanlon, A., and Murray, D.W., 1974. Time-Dependent Reinforced Concrete Slab Deflections, Journal of the Structural Division, ASCE, Vol. 100, No. ST9, September, pp. 1911-1924.

Murray, D.W. and Rajasekaran, S., 1975. Technique for Formulating Beam Equations, Journal of the Engineering Mechanics Division, ASCE, Vol. 101, No. EM5, October, pp 561-573.

Epstein, M. and Murray, D.W., 1976. Three-Dimensional Large Deformation Analysis of Thin Walled Beams, International Journal of Solids and Structures, Vol. 12, No. 12, December, pp. 867-876.

Epstein, M., Nixon, D., and Murray, D.W., 1978. Large Displacement Analysis of Beam-Columns, Journal of the Structural Division, ASCE, Vol. 104, No. ST5, May 1978, pp. 841-853.

Murray, D.W., 1979. A Review of Explosive Characteristics of Prestressed Secondary Containments Near Ultimate Load, Nuclear Engineering and Design, Vol. 52, pp. 157-164.

Elwi, A.E. and Murray, D.W., 1979. A 3D Hypoelastic Concrete Consitutive Relationship, Journal of the Engineering Mechanics Division, ASCE, Vol. 105, No. EM4, August, pp. 623-641.

Murray, D.W., 1979. Octahedral Based Incremental Stress- Strain Matrices, Journal of the Engineering Division, ASCE, Vol. 105, No. EM4, August, pp. 501-513.

Murray, D.W., Chitnuyanondh, L., Rijub-Agha, K.Y., and Wong, C., 1979. A Concrete Plasticity Theory for Biaxial Stress Analysis, Journal of the Engineering Mechanics Division, ASCE, Vol. 105, No. EM6, December, pp. 980-1006.

Murray, D.W., Chitnuyanondh, L., and Wong, C., 1980. Implementation of an Elastic-Plastic Concrete Relationship, Computer Methods in Applied Mechanics and Engineering, Vol. 23, pp. 35-57.

Murray, D.W., Simmonds, S.H., and MacGregor, J.G., 1981. Predicted Behavior of Gentilly-2 Containments, SMiRT6, Vol. J(a), Paper J 5/7, Paris.

Pinkney, R.B., Stephens, M.J., Murray, D.W., and Kulak, G.L., 1982. Inelastic Buckling of Axially Loaded Cylindrical Shells, Proceedings of the Canadian Society of Civil Engineering Annual Conference, May, pp. 169-186.

Scanlon, A. and Murray, D.W., 1982. Practical Calculation of Two-Way Slab Deflections, Concrete International, Vol. 4, No. 11, November, pp. 43-50.

El-Zanaty, M.H. and Murray, D.W., 1983. Nonlinear Finite Element Analysis of Steel Frames, Journal of the Structural Division, ASCE, Vol. 109, No. ST2, February, pp. 353-368.

Pinkney, R.B., Stephens, M.J., Murray, D.W., and Kulak, G.L., 1983. Use of Measured Imperfections to Predict the Buckling of Axially Loaded Cylindrical Shells, Canadian Journal of Civil Engineering, Vol. 10, No. 4, December, pp. 662-669.

Murray, D.W., 1984. Observations on Analysis, Testing and Failure of Prestressed Concrete Containments, Nuclear Engineering and Design, Vol. 82, pp. 37-46.

Elwi, A.E. and Murray, D.W., 1984. Nonlinear Analysis of Reinforced and Prestressed Containments, Proceedings on the International Conference on Computer Aided Analysis and Design, Split, Yugoslavia, Sept. 17-21, pp. 565-579.

Elwi, A.E. and Murray, D.W., 1985. Skyline Algorithms for Multilevel Substructure Analysis, International Journal for Numerical Methods in Engineering, Vol. 21, pp. 464-479.

Balakrishnan, S. and Murray, D.W., 1987. Prediction of Response of Concrete Beams and Panels by NLFEA, Proceedings of Colloquium Delft 1987 - Computational Mechanics of Reinforced Concrete, IABSE, August.

Balakrishnan, S. and Murray, D.W., 1988. Concrete Constitutive Model for NLFEA of Structures, Journal of Structural Engineering, ASCE, Vol. 114, No. 7, July, pp. 1449-1466.

Balakrishnan, S., Elwi, A.E., and Murray, D.W., 1988. Effect of Modelling on NLFEA of Concrete Structures, Journal of Structural Engineering, ASCE, Vol. 114, No. 7, July, pp. 1467-1487.

Balakrishnan, S. and Murray, D.W., 1988. Prediction of R/C Panel and Deep Beam Behavior by NLFEA, Journal of Structural Engineering, ASCE, Vol. 114, No. 10, October, pp. 2323-2342.

Balakrishnan, S. and Murray, D.W., 1988. Strength of Reinforced Concrete Panels, Canadian Journal for Civil Engineering, Vol. 15, No. 5, October, pp. 900- 911.

Napoleão, J., Elwi, A.E., and Murray, D.W., 1990. An Eigenvector Based Solution Strategy for Materially Nonlinear Problems, In FEM in the Design Process, ed. by John Robinson, Proceedings of the Sixth World Congress on Finite Element Methods, Bridestowe, England, October, pp. 705-714.

Napoleão, J., Elwi, A.E., and Murray, D.W., 1991. Eigenmode Dominance and its Application to Structures with Nonlinear Materials, Computers and Structures, Vol. 41, No. 2, pp. 281-291.

Behin, Z. and Murray, D.W., 1992. A Substructure-Frontal Technique for Erection Analysis of Cable-Stayed Bridges, Computers and Structures, Vol. 42, No. 2, pp. 145-157.

Napoleão, J., Elwi, A.E. and Murray, D.W., 1992. An Eigenvector Based Solution Strategy for Materially Nonlinear Problems. Computers and Structures, Vol. 42, No. 5, pp. 833-848.

Mohareb, M., Alexander, S.D.B., Kulak, G.L., and Murray, D.W., 1993. Laboratory Testing of Line Pipe to Determine Deformational Behavior, Proc. of the 12th Int'l Conf. on OMAE, Vol. V-Pipeline Technology, ASME, pp. 109-114.

Zhou, Z., and Murray, D.W., 1993. Behavior of Buried Pipelines Subjected to Imposed Deformations, Proc. of the 12th Int'l Conf. on OMAE, Vol. V-Pipeline Technology, ASME, pp. 115-122.

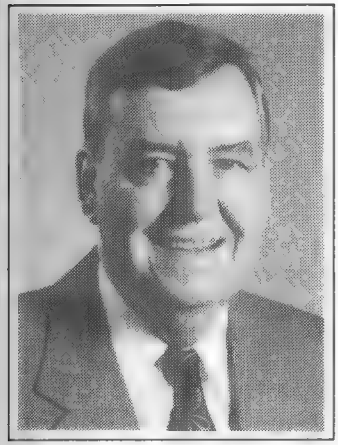
Zhou, Z., and Murray, D.W., 1993. Towards Rational Deformation Limit States for Buried Pipelines, Proc. of the 3rd ISOPE Conf., Vol II, pp. 18-24.

Yao, B. and Murray, D.W., 1993. Prediction of Distributed Discrete Concrete Cracking in RC Analysis, Journal of Structural Engineering, ASCE, Vol. 119, No. 10, pp. 2813-2834.

Souza, L.T. and Murray, D.W., 1993. An Alternative Pseudoperipheral Node Finder for Resequencing Schemes, International Journal for Numerical Methods in Engineering, Vol. 36, pp. 3351-3379.

GRADUATE STUDENTS SUPERVISED BY D.W. MURRAY

Name	Degree	Year	Thesis	Co-supervisor
Rajasekaran, S.	Ph. D.	1971	Analysis of Thin-Walled Beams	
Scanlon, Andrew	Ph. D.	1972	Time-Dependent Deflections of Reinforced Concrete Slabs	
Wu, F.	M. Sc.	1976	Decision Table Processing of CSA S16.1	
Srivastava, B.B.L.	M. Sc.	1976	Computation of Inelastic Instability in Steel Joists	
Elwi, Alaa E.	M. Sc.	1977	Nonlinear Analysis of Axisymmetric Reinforced Concrete Structures	
Kaliandasani, R.A.	M. Sc.	1977	Behavior of Open Web Steel Joists	Simmonds, S.H.
Matiisen, Rein A.	M. Sc.	1977	The Effects of Joint Eccentricity in Open Web Steel Joists	Simmonds, S.H.
Koziak, Basil D. P.	M. Sc.	1979	Analysis of Prestressed Wall Segments	
El-Zanaty, M.	Ph. D.	1980	Inelastic Behavior of Multistory Steel Frames	
Elwi, Alaa E.	Ph. D.	1980	Nonlinear Analysis of Axisymmetric Reinforced Concrete Structures	
Leigh, Edmond	M. Eng.	1982	Program for the Analysis of Non-Prismatic Frames	
Matthews, C. M.	M. Sc.	1982	Design of Floor Systems for Dynamic Response	Montgomery, C.J.
Ellis, Reed M.	M. Eng.	1984	Analysis and Design of Guyed Stacks and Towers	
MacGregor, Robert J.	M. Eng.	1984	Cable Supported Roof Enclosure for the Commonwealth Stadium in Edmonton, Alberta	
Balakrishnan, S.	Ph. D.	1986	Finite Element Prediction of Reinforced Concrete Behavior	
Behin, Zabihollah	Ph. D.	1990	Erection Analysis of Cable-Stayed Bridges	
Yao, Budan	Ph. D.	1992	Finite Element Analysis of Distributed Discrete Concrete Cracking	
Zhou, Zhilong	Ph. D.	1993	Numerical Structural Analysis of Buried Pipelines	



Sidney H. Simmonds

A Personal History

Sidney Herbert Simmonds was born in Winnipeg in 1931, the second child but eldest son of Sidney and Mary Simmonds. When he was five years old, the family moved to Sudbury, Ontario where he took his elementary schooling. After the family moved to Edmonton in 1943, he attended McKay Avenue and Garneau High schools. When Sid first enrolled at the University of Alberta it was in the Faculty of Arts with the intention of going into Law but he soon saw the light and transferred to Engineering. While at University, Sid became active in the Young People's Group at McDougall United Church. It was at this group that he met Elizabeth Anne Tilson and they were married on June 3, 1955.

Sid's first job after graduating in Civil Engineering in 1954 was with the City of Edmonton as a local improvement engineer. However, in October of that year, he joined the staff at the University of Alberta Buildings Office on the understanding that he could juggle hours to attend lectures in graduate courses. The Buildings Office (now known as Physical Plant) was a busy place in those days as the campus was beginning to expand and the staff consisted only of a half-time superintendent, two engineers and two draughtsmen. In addition to remodeling the aging campus buildings and the layout of new sidewalks and landscaping, the group was responsible for the design of several new buildings at the university farm, the layout for the extension of the then Agricultural Building to house geology and the biological sciences and for advising on the proposed new campus for the Calgary branch (later the University of Calgary). Sid found that he enjoyed graduate courses and although he had interests in both geotechnical and structural engineering and took the courses that enabled him to major in either area, his work at the buildings office led him to do his research in the area of structures. He received a M.Sc. degree in 1956.

While Sid was completing his M.Sc., Dr. George Ford convinced him to apply for an Athlone Fellowship. So, at the end of August, 1956, Betty and Sid together with their infant son, Douglas, sailed for England where he attended the University of Cambridge. Cambridge was an interesting place but housing, especially for a family with a new baby, was very difficult to find and they were lucky to get a one bedroom apartment in a house. About Christmas, Betty was not feeling well and it turned out that

she was again pregnant. With the housing situation and the fact that money was fast running out, they returned to Edmonton at the end of February, 1957. Their daughter, Patricia, was born in May of that year.

The first requirement after returning from England was to find a job. Sid went to the University and saw Dean Hardy who offered him an appointment as Assistant Professor of Applied Mechanics in the Department of Civil and Municipal Engineering (this was an eight month appointment with a salary that required you have other employment) and arranged a meeting with Tom Lamb, of Structural Engineering Services (later known as Lamb, McManus) for the other employment. Sid spent the next year (with the exception of the mornings after classes started when he gave 14 hours of lectures each week) designing the highway suspension bridge at Dunvegan and the spandrel arch concrete bridge on Saskatchewan Drive.

In 1958, all university academic positions became 12 month appointments. The next year the Department of Mechanical Engineering was established and Sid's title became Assistant Professor of Civil Engineering. However, there was still essentially no research involvement or graduate students to supervise and with the comparatively low salaries all staff had outside consulting employment. During the next three years, when he wasn't lecturing or marking, (junior staff did all their own marking including assignments) Sid worked as a design engineer with Associated Engineering Services Ltd. (AESL). Here he was involved mainly in designing water and sewage treatment facilities and arenas for hockey and curling throughout western Canada. It was during this period that he, with Hartley Daniels, proposed and did the preliminary design for a circular hockey arena for the City of Lloydminster which would have been a first. However, at the time the concept was considered too radical and the cost estimate not sufficiently reliable to depart from the traditional rectangular box and so it was never built.

By 1960 it had become apparent that there was going to be a much greater emphasis on research at universities across North America. At the University of Alberta this was emphasized by the selection of Dr. George Govier as Dean of the Faculty. Sid decided that it would be a good time to obtain further study and in September of that year, having obtained a leave of absence without pay, enrolled as a Ph.D. student at the University of Illinois. This exposed him to the world of the digital computer, Illiac II, paper tapes and programming in essentially an assembler language (punched cards and FORTRAN2 were introduced while he was completing his thesis). Sid completed his degree in August, 1962 and returned to the University of Alberta. Their third child, Gordon, was born in March, 1963.

During the sixties Sid became active on APEGGA committees including Counselling and Education, Publications and Meetings committees. During his four years on the Counselling and Education committee he represented the Association and frequently the U of A at more than 50 Career Days at High Schools throughout northern Alberta. As a member of the Meetings committee he was chairman of the arrangements subcommittee which looked after the accommodation, meals and entertainment for the Fiftieth Anniversary Annual Meeting held at Jasper Park Lodge.

The sixties was also a time of considerable expansion in western Canada. In addition to full-time work at the University, Sid continued to consult with AESL on

special projects. In 1966, he was part of the team that successfully put together the design-build proposal for Kaiser Resources in Sparwood, B. C. which was the first coal plant built to supply coal for shipment to Japan from Robert's Bank. As part of this project, Sid designed the first large coal silos in Canada to be used in continuous loading of unit trains. Later, also with AESL, he assisted in the concept and design of the hyperbolic roof shells over the clarifiers at the E. L. Smith Water Treatment Plant.

During the sixties, in the Faculty of Engineering, the marks of all first year students were monitored by what was known as the First Year Revisions Committee. After serving as chairman of this committee for several years, Sid was asked by Dean Hardy to act as Associate Dean, Academic during the illness and subsequent retirement of Leonard Gads who had held this position for many years. Sid continued in this position for nearly three years and still maintained a full teaching load. In the Fall of 1971, he requested that someone else be appointed with a reduced teaching load and returned to full-time in the Department of Civil Engineering.

In 1974, in cooperation with Drs. Murray and MacGregor, Sid became involved in a research project sponsored by the Atomic Energy Control Board of Canada. This involved developing a methodology for predicting the limit states under overpressure of Canadian prestressed concrete containment structures. The experimental part of this project continued until 1981. This led to a couple of consulting jobs for Quebec Hydro and in 1990 Sid prepared a report for Atomic Energy of Canada Ltd. on the condition of the secondary containments for the nuclear reactors in Cernavoda, Romania.

Sid took his first sabbatical leave in 1977-78 spending the first half as a visiting professor at the University of Texas, Austin and the second half at the University of South Carolina, Columbia. While in Columbia, Sid gave a graduate course under their APOGEE program which used satellite transmission so that he had students in thirteen cities in three states. During this period, the two older children moved into apartments with friends and attended university. While in Texas Gordon, who was then in Grade 9, acquired a full Texas accent and several Texas T-shirts. When he arrived in South Carolina with his Texas accent and T-shirts, it was not surprising that he was nick-named 'Tex' and had trouble convincing others that when he said he was going home it was not back to Texas.

The first half of Sid's second sabbatical was spent at the Royal Military College in Kingston, Ontario. The second half was spent in Edmonton but did include a lecture tour to five universities in southeastern United States. Sid's last sabbatical was in the first six months of 1992 which was spent in Switzerland, the first three months with VSL Research and Development Group in Bern and the next three months with the Swiss Federal Institute of Technology in Zurich. Weekends gave Betty and Sid a chance to see essentially all of Switzerland and two short holidays to see much of southern Germany and France.

Sid has always maintained active participation in engineering technical societies. He has served on many committees of the American Concrete Institute and, as the first President of the ACI Alberta Chapter, he conceived of the idea of a concrete toboggan race between Alberta engineering and technical schools that blossomed into the Great Northern Concrete Toboggan Race, with participants from all across Canada and the

United States and in the last two races from Europe. He is Secretary of CSA Committee A23.3 on Reinforced Concrete Design and chairman of the Two-way Slab subcommittee. He is a member of the Technical Affairs Committee, Structural Division of the Canadian Society for Civil Engineering and is the Chairman of the Organizing Committee for the CSCE Annual meeting in Edmonton in 1996. In recognition of his contributions, Sid has been elected a Fellow of the American Concrete Institute, of the American Society of Civil Engineers and of the Canadian Society for Civil Engineering and, most recently, was made a Life Member of APEGGA.

All three of their children are graduates of the University of Alberta, Douglas in Medicine, Patricia in Occupational Therapy and Gordon in Civil Engineering. Betty and Sid have six grandchildren that they are enjoying very much.

Sid took advantage of the University's early retirement programme and was appointed Professor Emeritus on July 1, 1993. Under the terms of his retirement he was able to retain use of his office for one year to complete research commitments and is co-supervising his final Ph.D. student. Betty and Sid are planning on remaining in Edmonton but hope to travel extensively, particularly in the wintertime. They are continuing to enjoy cross-country skiing and sailing, activities that they can enjoy with their grandchildren.

Professional Career Summary

Sidney H. Simmonds, Ph.D., P.Eng.

Dr. Simmonds is a structural engineer specializing in the behaviour and design of reinforced concrete structures. His special interests have been in the area of space enclosing structures such as continuous slabs, shell roofs, silos and containments. His research has been directed at a better understanding of the basic behaviour of structures and the results of some of this research has been incorporated in design Standards for reinforced concrete buildings. He has served on a number of technical committees including the committee responsible for developing the Canadian Standard for reinforced concrete design.

EDUCATION

B.Sc.	Civil Engineering (with distinction) University of Alberta	1954
M.Sc.	Civil Engineering, University of Alberta	1956
Ph.D.	Civil Engineering, University of Illinois	1962

EXPERIENCE

1970-present	Professor of Civil Engineering, University of Alberta
1963-1970	Associate Professor, Civil Engineering, University of Alberta
1960-1962	Graduate Student, University of Illinois
1957-1963	Assistant Professor, Civil Engineering, University of Alberta
1957-1960	Structural Design Engineer, Associated Engineering Services Ltd. (1958-1960), Structural Engineering Services (1957-1958).
1956-1957	Athlone Fellow, University of Cambridge, U.K.
1954-1956	Design Engineer, Buildings Office, University of Alberta
1954 (5 mths)	Local Improvement Engineer, City of Edmonton

For the past 32 years Dr. Simmonds has provided consulting services in the field of structural engineering to consulting engineers, contractors and government agencies in the form of specialized investigations, calculations and reports and in areas of litigation.

PROFESSIONAL MEMBERSHIPS

- Association of Professional Engineers, Geologists and Geophysicists of Alberta, Life Member
- Canadian Society for Civil Engineering, Fellow
- American Society of Civil Engineers, Fellow
- American Concrete Institute, Fellow
- International Association for Shell and Spatial Structures

TECHNICAL COMMITTEE MEMBERSHIPS

CSA Technical Committee on Reinforced Concrete Design A23.3

- Secretary (1984 - present)
- Subcommittee for Two-way Slabs (Chairman, 1984 - present)

Canadian Society for Civil Engineering

- Technical Affairs Committee - Structural Division
- Selection Committee for Gzowski Medal (Chairman, 1992-94)
- Organizing Committee for 1996 Annual Convention (Chairman)

American Concrete Institute

- ACI Committee 118 "Use of Computers" (Chairman, 1980-1984)
- ACI Committee 120 "History of Concrete" (Chairman, 1991-1995)
- ACI Subcommittee 318F "Code Committee for Two-way Slabs"
- ACI Committee 340 "Design Handbooks"

American Society of Civil Engineers

- ASCE/ACI Joint Committee 334 "Design of Shell Structures"
- ASCE/ACI Joint Committee 421 "Reinforced Concrete Slabs"

GRADUATE STUDENTS SUPERVISED

Name	Title	Year	Degree
Taylor, D.J.	Stress Strain Relationships of Lightweight Concrete	1960	MSc
Vollan, M.M.	Behavior of Cantilevered Flat-Plate Slabs	1964	MSc
Alexander, B.	Model Study of Hyperbolic Paraboloid Shell	1965	MSc
Rajendram, I.	Method of Analysis of Shallow Translational Shells	1965	MSc
Gyi, A.	Ultimate Strength and Behavior of Plates	1965	PhD
Kavanagh, R.J.	Influence of Column Shape on Flat Slabs	1967	MSc
Quapp, D.H.	Shallow Shells Supported by Elastic Beams	1969	MSc
Quinton, R.G.	Reinforced Concrete Cellular Orthotropic Slabs	1969	MSc
Smith, A.E.	Flat Plate on Columns Elongated in Plan	1969	MSc
Misic, J.	Comparative Study of Slab-Beam Systems	1970	MSc
Iyer, S.H.	Stiffness Coefficients for Cylindrical Shells	1970	PhD
Palko, F.	Deflection of Flat Slabs During Construction	1971	MEng
Waheed, A.	Design Moments by Equivalent Frame Method	1971	MEng
*Kaliandasani, R.	Behaviour of Open Web Steel Joists	1977	MSc

*Matiisen, R.	Effects of Joint Eccentricity in Steel Joists	1977	MSc
Shazly, A.	Arbitrary Loaded General Shells of Revolution	1978	MSc
Rogowsky, D.M.	Plastic Design of Reinforced Concrete Slabs	1980	MSc
Kerr, W.	Segment Method for Reinforced Concrete Slabs	1982	MSc
Heinrichs, P.W.	Reinforcement Patterns in Flat Plate Construction	1983	MEng
Malik, G.A.	Recommendations for Floor Framing Systems for the Instrument Hall for Telephone Exchange Building with Large Span (18 M) and Heavy Live Load (146 KN/M)	1983	MEng
Wong, W.S.C.	A Program for Reinforced Concrete Beam Design	1983	MEng
Hernandez, N.	Axisymmetrically Loaded Shells of Revolution	1983	MSc
Smith, A.B.B.	Lateral Coal Pressures in a Mass Flow Silo	1983	MSc
Cheng, T.F.	Slab Thickness and Moments Using Direct Design Method	1984	MEng
Chow, K.M.	A Structural Evaluation of a Tensegrity Dome	1984	MEng
Doblanko, B.W.	Shear-Moment Transfer Design Figures	1985	MEng
Playdon, D.K.	Finite Element Modelling of Buried Structures	1985	MSc
Shushkewich, K.	Analysis of Segmental Bridges	1985	PhD
Karkare, B.S.	Evaluation of Shear-moment Transfer in Flat Plates	1986	MEng
Alexander, S.D.B.	Shear-Moment Transfer in Slab-Column Connections`	1986	MSc
Small, G.E.	Microcomputer Spreadsheet Solution of Elastic Plate Bending Problems Using Finite Difference Operators and Iterative Techniques	1987	MEng
Bezzina, A.S.	Knowledge-Based CAD for the Design of Reinforced Concrete Columns	1987	MSc
Alexander, S.D.B.	Bond Model for Strength of Slab-Column Connections	1990	PhD
Fok, C.M.K.	Analysis of Spherical Domes with Variable Shell Thickness	1991	MEng
Elezaby, Y.K.	Modelling and Design of Unbraced Reinforced Concrete Frames	1992	PhD
Mulenga, M.N.	Frame Methods for Analysis of Reinforced Concrete Two-Way Slabs	1993	PhD

* Jointly supervised with D.W. Murray

SUMMARY OF TECHNICAL PAPERS, SINCE 1984

Technical Journals

Alexander, S.D.B. and Simmonds, S.H., 1992. Column-Flat Plate Connection Tests, ACI Structural Journal, V.89, No. 5, Sept-Oct., pp. 495-502.

Alexander, S.D.B. and Simmonds, S.H., 1992. Punching Shear Tests of Concrete Slabs Column Joints Containing Fiber Reinforcement, ACI Structural Journal V89, No. 4, July-August, pp. 425-432.

Alexander, S.D.B. and Simmonds, S.H., 1992. Bond Model for Concentric Punching Shear, ACI Structural Journal V 89, No. 3. May-June, pp. 325-334.

Small, G.E. and Simmonds, S.H., 1990. Spreadsheet Solutions of Elastic Plate-Beam Problems, Canadian Journal of Civil Engineering, Vol. 17, No. 6, December, pp. 931-939.

Bezzina, A.S. and Simmonds, S.H., 1989. Knowledge-Based Expert Systems in Reinforced Concrete Design, Concrete International: Design and Construction, Vol. 11, No. 2, February, pp. 57-61.

Simmonds, S.H., 1989. Effect of Support Movement on Hyperbolic Paraboloid Shells, Journal of the Structural Division, ASCE, Vol. 115, No. 1, January pp. 19-31.

Bezzina, A.S. and Simmonds, S.H., 1988. COLUMN: A KBES for the Design of Reinforced Concrete Columns, American Concrete Institute, Computer Applications in Concrete Technology SP-111, Detroit, December, pp. 17-32.

Alexander, S.D.B. and Simmonds, S.H., 1988. Shear-Moment Interaction of Slab-Column Connections, Canadian Journal of Civil Engineering, Vol. 15, No. 5, October, pp. 828-833.

Playdon, D.K. and Simmonds, S.H., 1988. Behaviour of Slab-Stiffened Culvert Structures, Canadian Journal of Civil Engineering, Vol. 15, No. 4, August, pp. 726-731.

Simmonds, S.H. and Playdon, D.K., 1988. Modelling Soil-Structure Interaction Construction, Computers and Structures, Vol. 28, No. 2, February, pp. 283-288.

Simmonds, S.H. and Alexander, S.D.B., 1987. Model for Column-Slab Connections, Structural Journal of the American Concrete Institute, Vol. 84, No. 4, July-Aug., pp. 296-303.

Alexander, S.D.B. and Simmonds, S.H., 1987. Ultimate Strength of Slab-Column Connections, Structural Journal of the American Concrete Institute, Vol. 84, No. 3, May-June, pp. 255- 261.

Simmonds, S.H. and Doblanko, B.W., 1986. Shear Moment Transfer in Flat Plates, Canadian Journal of Civil Engineering, Vol. 13, No. 3, June, pp. 327-334.

Rizkalla, S.H., Simmonds, S.H., MacGregor, J.G., 1984. Cracking of P/C Nuclear Containment Structures Journal of the Structural Division, ASCE, Vol. 110, No. 9, Sept., pp. 2148-2163.

Rizkalla, S.H., Lau, B.L., Simmonds, S.H., 1984. Air Leakage Characteristics in Reinforced Concrete, Journal of the Structural Division, ASCE, Vol. 110, No. 5, May, pp. 1149-1162.

Rizkalla, S.H., Simmonds, S.H., MacGregor, J.G., 1984. Prestressed Concrete Containment Model, Journal of the Structural Division, ASCE, Vol. 110, No. 4, April, pp. 730-743.

Conference Proceedings

Elezaby, Y.K. and Simmonds, S.H., 1992. An Expert System for the Design of Unbraced Reinforced Concrete Frames, Proceedings of the Second Canadian Conference in Computing in Civil Engineering, Ottawa, August 1, pp. 266-276.

Alexander, S.D.B. and Simmonds, S.H., 1991. Tests of Column - Flat Plate Connections, Proceedings, Canadian Society for Civil Engineering, May, Vol. 3, pp. 1-10.

Alexander, S.D.B. and Simmonds, S.H., 1991. Board Model for Punching Strength of Slab - Column Connections, Proceedings, 1AB SE Colloquium Stuttgart, May, pp. 709-714.

Simmonds, S.H. and Elezaby, Y.K., 1990. Some Thoughts on KBES for Structural Design, Proceedings of the Forth Rail Bridge Centenary Conference, Edinburgh, August, pp. 1338-1347.

Dick, J.E., Simmonds, S.H. and Horshchel, D.S., 1990. Crack Leakage Equipments for Concrete Containers, Proceedings of the American Nuclear Society, Nashville, June, pp. 243-244.

Small, G.E. and Simmonds, S.H., 1989. Spreadsheet Solutions of Elastic Plate-Beam Problems, Canadian Society for Civil Engineering, Proceedings Annual Conference, St. John's, Newfoundland, June, Vol. 3, pp. 121-133.

Simmonds, S.H. and Bezzina, A.S., 1988. KBES for Proportioning Reinforced Concrete Members, Proceedings of the Third International Conference on Computing in Civil Engineering, Vancouver, August, pp. 127-132.

Small, G.E. and Simmonds, S.H., 1988. Spreadsheet Analysis of Plate Bending Problems, Proceedings of the Third International Conference on Computing in Civil Engineering, Vancouver, August, pp. 37-44.

Playdon, D.K. and Simmonds, S.H., 1987. Modelling Soil Placement in Soil-Structure Interaction Problems, Proceedings of 11th Canadian Congress of Applied Mechanics, Edmonton, June, pp. 234-235.

Alexander, S.D.B. and Simmonds, S.H., 1987. Shear-Moment Interaction of Slab-Column Connections, Canadian Society for Civil Engineering, Proceedings Annual Conference, Montreal, May, pp. 555-571.

Playdon, D.K. and Simmonds, S.H., 1986. Behavior of Buried Slab-Culvert Structures, Canadian Society for Civil Engineering, Proceedings Annual Conference, Toronto, May, 25 pp.

Playdon, D.K. and Simmonds, S.H., 1984. Finite Element Modelling of Buried Structures, Symposium on Advances and Trends in Structures and Dynamics, NASA Conference Publication 2335, Washington, D.C., October, pp. 395-404.

Contributions to Handbooks

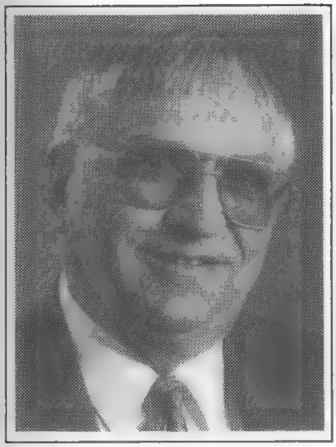
SP-17(91), 1991. ACI Design Handbook - Vol. 1, American Concrete Institute, Detroit, 361 pp.

SP-17(91)(S), 1991. Design of Two-way Slabs, Supplement to ACI Design Handbook - Vol. 1, American Concrete Institute, Detroit, 104 pp.

SP-17A(90), 1990. ACI Design Handbook - Vol. 2, American Concrete Institute, Detroit, 221 pp.

Simmonds, S.H., 1985. CPCA Concrete Design Handbook, Chapter 5, Slabs, pp. 5-1 to 5-59, Canadian Portland Cement Association, Ottawa, Dec.

Simmonds, S.H., 1985. Handbook of Concrete Engineering - Second Edition, Edited by M. Fintel, Chapter 3, One- and Two-Way Slabs, pp. 81-120, Van Nostrand Reinhold Company, New York.



Joseph Warwaruk

A Personal History

Dr. Joseph Warwaruk, Professor Emeritus of the Civil Engineering Department at the University of Alberta, was born in Edmonton Alberta on September 19, 1930. He grew up in the farming community of Waugh, north of Edmonton, and completed his high school education at Legal Alberta. He married Alexandra Hukalo in 1956. They have one daughter, Susanne, and two sons, David and Mark.

He attended the University of Alberta following high school and received a Bachelor of Science degree with distinction in 1954. In September 1955 he began post-graduate studies at the University of Illinois at Urbana–Champaign, USA. In 1957 he completed a Master of Science degree and a Doctor of Philosophy degree in 1960. He studied under the guidance of Professor Chester Siess and Professor Mete Sozen for both degrees. His area of specialization was prestressed concrete behavior with emphasis on the flexural strength and behavior of beams.

In September 1957 he began his academic career as an Instructor in the Department of Applied Mechanics at the University of Illinois. In September 1960 he was appointed Assistant Professor in the Department of Civil Engineering at the University of Alberta and remained in the department until his retirement in December 1991. He was promoted to Associate Professor in 1962 and Professor of Civil Engineering in 1969.

A sabbatical year leave opportunity in 1971–72 permitted him to spend a semester as a Visiting Professor at West Virginia University in Morgantown West Virginia and a six month period as a Research guest worker at the U.S. National Bureau of Standards in the Washington D.C. area studying aspects of structural behavior of housing systems.

Throughout his academic career he instructed in subject areas of Statics, Dynamics, Mechanics of Deformable Bodies, Surveying, Computer Programming, Structural Analysis, Design in Wood, Reinforced Concrete, Prestressed Concrete, Reinforced Structural Masonry, and Technical Report Writing at the undergraduate level and in subject areas of Structural Analysis, Behavior of Reinforced Concrete Members, Design of Reinforced Structural Masonry, and Behavior of Prestressed Concrete Members at the graduate level.

Research activity during his academic career focused on structural problems of a practical nature with details directed to improvement and development of design procedures and codes, but always with a clear sense of responsibility to the education of graduate students who were intimately associated with such research. Early efforts with other members of the Structures group involved design and construction of the I.F. Morrison Structural Engineering Laboratory which opened in 1964. Areas of research involved reinforced and prestressed concrete under a variety of different loads, and more recently, various aspects of concrete and clay brick masonry structural wall and column elements. Specifics are given in the list of publications. He served on Technical Committees of the American Concrete Institute dealing with torsion and shear in reinforced concrete members and of the Canadian Standards/National Building Code dealing with limit states design of masonry buildings. In 1983, together with Professor Jack Longworth, he participated in organizing and successfully executing the Third Canadian Masonry Symposium held in Edmonton.

At the University he served on numerous committees. Within the Department these included Academic Planning and Curriculum, Laboratory Operations, Budget, Infrastructure and Graduate Students. In the Faculty of Engineering among others were Engineering Executive Coordinating Committee, Academic Appeals, Salaries and Promotion, Member Selection, Computer Aided Instruction, Faculty First Year Computing Course. In General Faculties Council these included Radio and Television, Computer Facilities and Policy, member of GFC from Engineering, Executive Committee of GFC and GFC Salaries and Promotion Committee. He also was a member of academic appeals, coordinating council, and executive member of coordinating council for the Association of Academic Staff.

Professional Career Summary

Joseph Warwaruk, Ph.D., P.Eng.

PERSONAL DATA

Position:	Professor of Civil Engineering University of Alberta Edmonton, Alberta T6G 2G7
Nationality:	Canadian
Date of Birth:	September 19, 1930 Married, three children

EDUCATION

B.Sc.	Civil Engineering (With Distinction)	University of Alberta	1954
M.S.	Civil Engineering	University of Illinois	1957
Ph.D.	Civil Engineering	University of Illinois	1960

ACADEMIC EXPERIENCE

Instructor	Applied Mechanics	University of Illinois	1957-1960
Assistant Professor	Civil Engineering	University of Alberta	1960-1962
Associate Professor	Civil Engineering	University of Alberta	1962-1969
Visiting Professor	Civil Engineering	West Virginia University	Fall 1971
Professor	Civil Engineering	University of Alberta	1969-Present

RESEARCH EXPERIENCE

1955-1957	Research Assistant in the Department of Civil Engineering, University of Illinois
1958-1960	Research Assistant in the Department of Civil Engineering, University of Illinois - 4 months each summer
1960-Present	Supervising graduate students at the University of Alberta. Supervised four Research Associates: W.R. Hibbard, J. Misic, R. Linder, and R. Lee, and one P.D.F., Dr. P. Mukherjee.
1972 Jan-June	Research guest worker at the U.S. National Bureau of Standards, Washington, D.C.

TEACHING EXPERIENCE

1957-1960	Instructing in the Department of Theoretical and Applied Mechanics in subject areas of Statics, Dynamics, Strength of Materials, and Strength of Materials Laboratory
1960-Present	Instructing in the Department of Civil Engineering in subject areas of Statics, Dynamics, Mechanics of Deformable Bodies, Surveying, Computer Programming, Structural Analysis, Design in Wood, Reinforced Concrete, Prestressed Concrete Reinforced Masonry, Technical and Report Writing Advanced Structural Analysis and Advanced Strength of Materials in the undergraduate program, and, in subject areas of Structural Analysis, Behavior of Reinforced Concrete Members, Design of Reinforced Masonry, and Design and Behavior of Prestressed Concrete Members at the graduate level.

RESEARCH GRANTS

1962	General Research #359	$525	Bond and Splitting Stresses in Reinforced Concrete
1963	General Research #359	$300	Bond and Splitting
1963	National Research Council #A-1696	$2000	Resistance of Concrete to Torsion
1964	NRC #A-1696	$2800	Resistance of Concrete to Torsion
1965	NRC #A-1696	$4000	Prestressed Concrete Studies
1966	NRC #A-1696	$8000	Torsion and Prestress Studies
1967	NRC #A-1696	$10,000	Torsion and Prestress Studies
1968	NRC #A-1696	$12,740	Combined Loading in Reinforced and Prestressed Concrete
1969	NRC #A-1696	$15,000	Combined and Limit Loads for Concrete Beams
1970	NRC #A-1696	$15,000	Torsion in Prestressed Concrete Beams
1971-74	NRC #A-1696	$16,000	per year for three years - Studies in Prestressed Concrete
1974-77	NRC #A-1696	$14,000	per year for three years - Prestressed Concrete Investigations
1977-80	NRC #A-1696	$17,500	per year for three years - Research in Prestressed Concrete and Masonry
1980-83	NSERC #A1696	$23,650	per year for three years - Research in Prestressed Concrete and Masonry

1983-86	NSERC #A1696	$31,721	per year for three years - Research in Prestressed Concrete Structural Masonry
1983	NSERC #E7053	$130,650	Major Equipment Grant for Data Acquisition and Control System with others in the Structural Engineering Group
1977-84		$10,000	per year from the Alberta Masonry Institute and the Canadian Masonry Research Council for Research in Masonry
1985-86	AMI-DOW	$39,000	Contract research for AMI-DOW to study structural effectiveness of rigid SM insulation in masonry veneer wall systems.
1986	Cross-Country Insulation Fund	$8,000	Research on contribution of rigid and semirigid insulation to structural strength of composite walls.
1987-90	NSERC A1696	$16,000	per year - Research in Masonry Structures
1989	Prairie Masonry Research Institute	$21,000	Shear Transfer Connector Studies
1989	Municipal Affairs Gov't of Alberta	$20,500	Thin Wall Concrete Foundations
1990	NSERC A1696	$12,000	per year - Research in Masonry Structures
1991	Prairie Masonry Research Institute	$16,000	Shear Transfer Connector Studies

INDUSTRIAL EXPERIENCE

1950-1954	During summer periods experience gained in surveying and materials inspection while employed in the Alberta Department of Highways.
1954-1955	Bridge Engineer with Bridge Branch of the Alberta Department of Highways.
Summer 1955	Resident Bridge Engineer for R.M. Hardy and Associates Ltd.
Summer 1956	Structural Designer for Stanley, Grimble and Roblin Ltd.
1955-1960	Research Assistant in the Civil Engineering Department of the University of Illinois
1972	Guest Worker in Structures Section of the Center for Building Technology, National Bureau of Standards, January to June 1972, Washington, D.C.
1960-Present	Involved in the solution of specialized problems for consulting firms in the area of bridge and building construction.

CONTINUING EDUCATION

Summer 1961	Attended Ford Foundation sponsored course at Michigan dealing with Computers in Engineering Education.
Summer 1965	Attended Earthquake Engineering short course at University of British Columbia.
Summer 1966	Attended course on the Integrated Civil Engineering System at M.I.T.
Summer 1967	Attended short course involving resistance of tall buildings to earthquakes at Seattle and also attended an NSF sponsored course on Computer Programming for Structural Engineering at Stanford.
Summer 1968	Attended two week course on Industrialized System Building at M.I.T.
January 1970	Attended Systems Building Research Seminar sponsored by American Society of Civil Engineers at Asilomar California.
Summer 1972	Attended Conference on Industrialized Building at West Virginia University sponsored by the National Science Foundation and the American Society of Civil Engineers. Served on Conference Steering Committee.
Summer 1975	Attended seminar in Reinforced Masonry at McMaster University.
Summer 1986	Attended seminar sponsored by the Canadian Prestressed Concrete Institute for University Professors dealing with metric prestressed concrete design.

EXTENSION ACTIVITIES

1961	Participated in Structural Engineering Conference held at the University of Alberta
1965	Participated in Seminar in Computing Science for Engineers at U. of A.
1967	Non-credit evening course on Fundamentals of Computer Programming for Engineers in fall and spring terms
1967	Structural Analysis using Newmark's Method in spring term
1968	Noncredit evening course in Fundamentals of Computer Programing for Engineers in both fall and spring terms
1969	Noncredit evening course in Fundamentals of Computer Programming for Engineers, in spring term
1970	Noncredit evening course in Computer Programming for Engineers, fall term, shared with Dr. G. Fisher
1971	Participated in Seminar on Building Code Requirements ACI 318-71

1972 Participated in Evening Seminar on Reinforced Concrete Building Code, November 1972

1977 Seminar presentation on Masonry Design and Construction

1980 Participated in Seminar "Background of Requirements for Design in Torsion" sponsored by the American Concrete Institute.

1983 Organized with Jack Longworth: The Third Canadian Masonry Symposium '83 held in Edmonton Alberta June 1983.

COMMITTEE ACTIVITIES

Department of Civil Engineering:

Served on various committees dealing with Nominations, Curriculum, Academic Planning, Student Registration and Counselling, Planning, Equipment Purchasing, Laboratory Operations, Budget, Infrastructure, Graduate Students.

Faculty of Engineering:

Served on committees involving Curriculum, Third Year Revisions, Radio and Television, Physical Planning, Engineering Executive Coordinating Committee, Academic Appeals, Salary and Promotions, Student Work Employment Program, Selection Committees for Chairman, Ad Hoc Committee on Computer Aided Instruction, Ad Hoc Committee on First Year Computing Course for Engineering Students.

General Faculty Council:

Radio and Television, Computer Facilities and Policy, Engineering Faculty Council member, General Faculty Council member, member of Executive Committee of General Faculties Council, member of General Salaries and Promotions Committee.

Association of Academic Staff, University of Alberta:

Academic Appeals Committee, Coordinating Council, and Executive member of Coordinating Council

Association of Professional Engineers, Geologists and Geophysicists of Alberta:

Membership Committee, Seconded to APEGGA Committee on Engineering Education, Board of Engineering Education, Engineering Internship, Student Work Employment.

American Concrete Institute:

Member of Committee 438, Torsion, dealing with formulation of design recommendations for combined loadings and, member of Committee 115, Research, Corresponding member Committee ACI-ASCE 445 Shear and Torsion, member of Board of Directors, Alberta Chapter ACI.

Canadian Standards Association and National Building Code:

Member of Subcommittee on Programming Languages of the Committee on Computer and Information Processing and, CSA/NBC Committee on Torsion and Shear Design, member of Task Force Committee S304 Masonry Design and Construction for Buildings.

Northern Alberta Institute of Technology

Member on Advisory Committee for Civil Engineering Technology.
Student Chapter CSCE
Faculty Representative and Advisor.

SELECTED PUBLICATIONS

(Total publications 1960 to present is 76.)

Warwaruk, J., Sozen, M.A. and Siess, C.P., 1962. Strength and Behaviour of Flexure of Prestressed Concrete Beams, University of Illinois, Engineering Experiment Station Bulletin No. 464.

Warwaruk, J., 1962. Flexural Strength of Prestressed Concrete Beams using a Graphical Solution for the Reinforcement Stress, Transactions E.I.C., Vol. 5, No. 4.

Warwaruk, J., 1965. Deformation Analysis for Prestressed Concrete Beams, Journal of the Prestressed Concrete Institute, Vol. 10, No. 5, Presented also at annual meeting of PCI.

Ragan, H.S. and Warwaruk, J., 1967. Tee Members with Large Openings,, Journal of the Prestressed Concrete Institute, Vol. 12, No. 4, pp. 52-65.

Pandit, G.S. and Warwaruk, J., 1968. Reinforced Concrete Beams in Combined Bending and Torsion, Paper SP-18-5 of Publication of the American Concrete Institute, SP-18 Torsion of Structural Concrete.

McMullen, A.E. and Warwaruk, J., 1970. Concrete Beams in Bending, Torsion, and Shear, Journal of the Structural Division, Proceedings ASCE, Paper No. 7270.

Mukherjee, P.R. and Warwaruk, J., 1971. Torsion, Bending and Shear in Prestressed Concrete, Paper No. 1063, Proceedings, American Society of Civil Engineering.

Warwaruk, J., 1974. Behavior of Prestressed Concrete T-Beams with Large Rectangular Web Openings, Shear in Reinforced Concrete, Publication SP- 42, American Concrete Institute, Detroit, pp. 399-424.

McMullen, A.E. and Warwaruk, J., 1974. Strength of Concrete Beams with Web Reinforcement under Combined Loading, Transactions of the Canadian Society for Civil Engineering, EIC Vol. 17, No. A-4, pp. 1-7.

Misic, J. and Warwaruk, J., 1978. Strength of Prestressed Solid and Hollow Beams Subjected Simultaneously to Torsion, Shear and Bending, Paper published in the Douglas McHenry International Symposium on Concrete and Concrete Structures, American Concrete Institute, Publication SP-55, Detroit, pp. 515-545.

Hatzinikolas, M., Longworth, J., and Warwaruk, J., 1978. The Effect of Joint Reinforcement on Vertical Load Carrying Capacity of Hollow Block Masonry, Proceedings of the North American Masonry Conference, Boulder, Colorado, pp. 16-1 to 16-16.

Hatzinikolas, M., Longworth, J., and Warwaruk, J., 1978. Buckling of Plain Masonry Walls with Initial Double Curvature, Proceedings of the North American Masonry Conference, Boulder, Colorado, pp. 87-1 to 87-14.

Warwaruk, J., 1979. Deflection Requirements - History and Background Related to Vibrations, Vibrations of Concrete Structures, American Concrete Institute, Publication SP-60, Detroit, pp. 13-41.

Hatzinikolas, M., Longworth, J., and Warwaruk, J., 1980. The Analysis of Eccentrically Loaded Masonry Walls by the Moment Magnifier Method, Proceedings of the Second Canadian Masonry Symposium, Ottawa, Canada, pp. 245-258.

Hatzinikolas, M., Longworth, J., and Warwaruk, J., 1980. Failure Modes for Eccentrically Loaded Concrete Block Masonry Walls, Journal of the American Concrete Institute, No. 4, V. 77, pp. 258-263.

Hatzinikolas, M., Longworth, J., and Warwaruk, J., 1980. Evaluation of Tensile Bond and Shear Bond of Masonry by Means of Centrifugal Force, Canadian Journal of Civil Engineering, Vol. 7, No. 1, pp. 192-197.

Taylor, G.W. and Warwaruk, J., 1981. Strength and Deformation of Pretensioned Box Girders, Journal of the Structural Division American Society of Civil Engineering, Vol. 107, No. ST5, Paper No. 16252, pp. 775-788.

Warwaruk, J., 1981. Torsion in Reinforced Concrete, Significant Developments in Engineering Practice and Research, American Concrete Institute, Special Publication, No. 72, pp. 247-277.

Hatzinikolas, M., Longworth, J., and Warwaruk, J., 1982. Corrugated Strip Ties in Curtain Wall Construction, Proceedings of the Second North American Masonry Conference, University of Maryland, College Park, Md. U.S.A., pp. 14-1 to 14-14.

Lee, R., Longworth, J., and Warwaruk, J., 1983. Behaviour of Restrained Masonry Beams, Proceedings of the Third Canadian Masonry Symposium, Edmonton, Canada, June, pp. 37-1 to 37-16.

Sturgeon, G.R., Longworth, J. and Warwaruk, J., 1985. Reinforced Concrete Block Masonry Columns, Proceedings of the Third North American Masonry Conference, University of Texas, Arlington, Texas, June, pp. 20-1 to 20-16.

McGinley, W.M., Warwaruk, J., Longworth, J. and Hatzinikolas, M., 1986. Masonry Veneer and Steel Stud Curtain Walls, Proceedings of the 4th Canadian Masonry Symposium, University of New Brunswick, Fredericton, N.B., June, pp. 730-743.

McGinley, W.M., Warwaruk, J., Longworth, J. and Hatzinikolas, M., 1987. Load-Deflection Behaviour of Curtain Wall Backing Systems, Proceedings of the Fourth North American Masonry Conference, Los Angeles, August.

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McGinley, W.M., Warwaruk, J., Longworth, J. and Hatzinikolas, M., 1989. Limit States Design of Masonry Veneer Wall Systems, Proceedings of the Fifth Canadian Masonry Symposium, Vancouver, B.C., June, pp. 111-124.

Papanikolas, P.K., Hatzinikolas, M.A., Warwaruk, J., and Elwi, A.E., 1989. Experimental and Analytical Results for Shear Connected Cavity Walls, Proceedings of the Fifth Canadian Masonry Symposium, Vancouver, B.C., June, pp. 251-261.

Ungstad, D.G., Hatzinikolas, M.A., and Warwaruk, J., 1990. Prestressed Concrete Masonry Walls, Proceedings of the Fifth North American Masonry Conference, Urbana-Champaign, Illinois, U.S.A., June, pp. 1147-1162.

Muqtadir, M.A., Warwaruk, J., Hatzinikolas, M.A., and Longworth, J., 1990. Slenderness Effects in Eccentrically Loaded Plain Masonry Walls, Proceedings of the Fifth North American Masonry Conference, Urbana-Champaign, Illinois, U.S.A., June, pp. 1007-1016.

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Goyal, A.J., Warwaruk, J., and Hatzinikolas, M.A., 1992. Shear Connected Cavity Walls Under Eccentric Loading, Proceedings of the 6th Canadian Masonry Symposium, Saskatoon, Saskatchewan, June, pp. 47-59.

Dozzi, S.P., Warwaruk, J., and Lavoillette, B., 1992. Thin Unreinforced Concrete House Foundation Walls, Journal of the Canadian Society of Civil Engineering, June, pp. 389-395.

Goyal, A., Hatzinikolas, M.A., and Warwaruk, J., 1993. Influence of Shear Connected Masonry Veneer on Eccentrically Loaded Reinforced Cavity Walls, Sixth North American Masonry Conference, Philadelphia, PA, June, pp. 1079-1090.

TECHNICAL PAPERS

THE HISTORY AND FUTURE OF LIMIT STATES DESIGN

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Ottawa, Ontario
Canada, K1A 0R6

ABSTRACT

This article discusses limit states design as it relates to practical thinking, provides a brief history of its development, and finally discusses future developments such as limit states design for durability and for the design of materials.

INTRODUCTION

The motivation for this paper is the happy association the author has had in the development of limit states design in Canada with professors of the University of Alberta who are retiring, plus the fact that the author is also going to retire (but not stop working) within the next year.

THINKING

Like the professors who are retiring, I have always enjoyed doing research. Basically I want to learn something about the world I live in, and tell it to others in a way that will be *useful* to them. Einstein said that *'research is the application of everyday thinking'*, but this is also true of engineering practice or any other human activity. If we can think better, then we can do better, both in practice and in research. Limit states design is directly related to the way we think. The following discussion is based on two pocket books by DeBono (1971, 1976).

The basic biological purpose of thinking is to enable a living organism (including a human) to survive by getting the things that it needs and keeping clear of the things that are dangerous. This is what motivates our thinking. It is basically a matter of knowing what to do about a situation that confronts us. For humans the basic thinking processes are:

- instinct - a prewired reaction to a situation
- learning - do/don't rules based on experience
- *understanding*

Understanding is a thinking process that changes a new and unfamiliar situation into one or more familiar ones, which we can handle. Understanding is the basis of all scientific and engineering theories and models. One model is the free body diagram based on Newton's law of statics. Another model, load transfer, is illustrated in Figure 1. A third model, a model of how the mind works, is described in Figure 2. Being a construction of the human mind, the models never perfectly represent reality. But if they contain the essential features, and are simple to reconstruct in the mind, they are very useful.

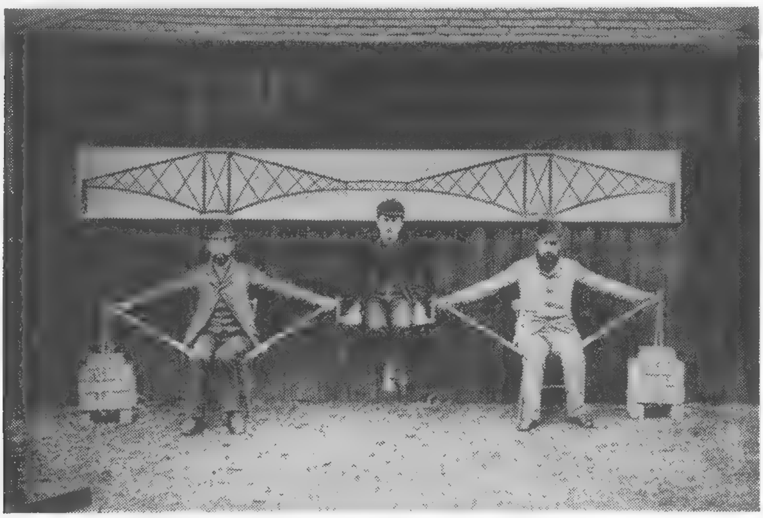
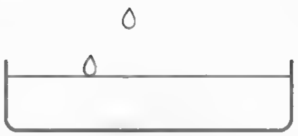


FIG. 1. A Living Model of the Forth Bridge

Understanding is extremely powerful because it explains and predicts what happens based on a few generic ideas. The ideas or models, however, must be based on experience. As a profession we must understand why structures fail. And the way we understand why they fail is by the use of models - ideas we can manipulate. It is these ideas by which we explain why a structure will or will not fail. It is these ideas we use to explain to others why something will or will not work. Understanding is therefore a powerful tool for communication. Lack of communication is behind most of the failures that occur in the practice of structural engineering. The book of Genesis, Chapter 11, describes the failure of the building of the city of Babel, including a very high tower (Figure 3), was caused by lack of communication because the Lord *'confounded their language, that they may not understand one another's speech'*. Hence the word *'babble'*.

Jelly Model



A dish of jelly initially has a flat surface, onto which drops of water are spooned in a pattern sequence. The water melts the jelly forming depressions before evaporating. Water subsequently spooned onto or adacent to a depressed area flows into it and deepens it. Accumulated water in a depression flows into a deeper depression, and eventually a well established system of flow is carved out. Even when no water falls on the surface, there is a flow of waterdroplets in well-established directions.

FIG. 2. A Model of the Mind

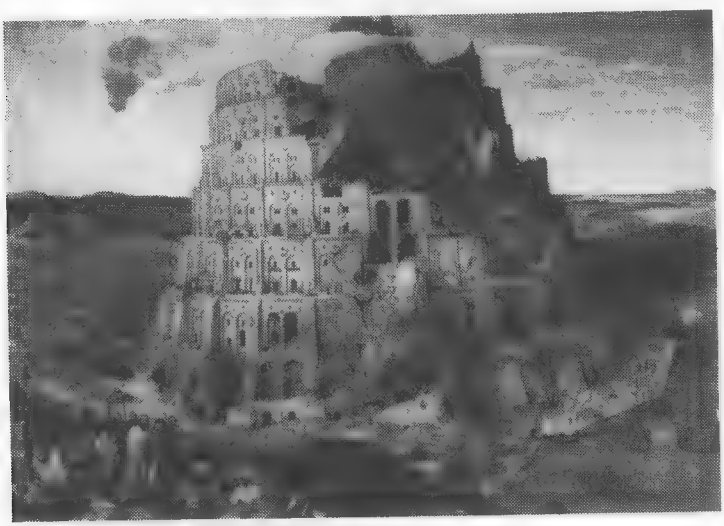


FIG. 3 Tower of Babel (painting by Breughel)

Learning (do/don't rules) based on experience is another important thinking tool, and prior to Newton, was the only way to design new structures, essentially by trial and error. My feeling is, however, that early builders made use of concepts such as statics (load transfer), at least qualitatively.

LIMIT STATES DESIGN

The basic biological purpose of thinking is to survive by getting the things that we need (structures for different purposes) and to avoid the things that are dangerous or make life difficult (failures of various kinds). The limit states define the things that are dangerous (ultimate limit states) and the things that make life difficult (serviceability limit states). Newton's Laws of statics and dynamics, testing and theories of structural behaviour (e.g. elastic theory) provide the means of calculating whether the structure will fail and how. This gives the engineer an understanding useful for practice. Safety factors (load and resistance factors) are incorporated to keep the risk due to the uncertainty in the calculation assumptions sufficiently low. In reality, to make the method practical, limit states design is not quite as realistic as this:

Limit States: It is true that the limit states are based on the things that go wrong but they also must be predictable, which usually means calculable. For example exceeding strength may mean collapse or it may mean local damage. Exceeding a deflection limit does not necessarily mean failure unless it causes damage of non-structural components, gaps which allow leakage, or poor drainage. Limit states design makes use of strength and deflection because they are calculable. The more complex failure phenomena of collapse, damage, leakage, etc. are not usually directly calculable, consequently we use strength and deflection as primary indicators of safety and serviceability failures and, by experience and testing, determine critical values for design and evaluation. Better practical theories help improve the connection to reality; for example the vibration limit state for human annoyance is now expressed in terms of an acceleration limit rather than a deflection limit. The limit states are categorized as follows:

Ultimate Limit States (safety):
- loss of equilibrium (overturning, sliding, etc.)
- structural failure (exceeding the strength of components or parts of the structure)

Serviceability Limit States (performance of structure or its appearance):
- local damage (cracking, spalling, slip of components)
- excessive displacement
- excessive vibration

Theories or Models: Although Newton's Laws are essentially unchanged for civil engineering practice, in recent years theories of structural behaviour (mechanics of materials) have broadened to include plastic or slip theory as well as elastic theory, or a convenient mixture of them. Many simplifying assumptions are used to obtain practical formulae for design. It is important, however, that the formulae reflect the essence of the structural behaviour so that intelligent designs are obtained.

Calculations: Calculations range all the way from mental estimates and back-of-the-envelope calculations to finite element calculations, including the determination of dynamic response. In all cases, the essence of structural behaviour under applied load

should be self-evident (transparent). Simple calculations are preferred for preliminary design and checking, because they tend to generate more thinking (modelling) about the structure than computer calculations. All limit states design calculations are carried out to satisfy the following criteria:

Ultimate Limit States:

[1]	Load Effect (force)	<	Resistance

Serviceability Limit States:

[2]	Load Effect (deflection, strain, acceleration, etc.)	<	Limit indicating onset of serviceability failure

Risk: Load and resistance factors are introduced into the calculations to keep the risk due to uncertainty in the calculation assumptions sufficiently low. The greater the consequences of failure, the lower should be the probability of failure. Ultimate limit states therefore require greater safety margins than the serviceability limit states. In addition there are greater uncertainties associated with the serviceability limit states, especially regarding the definition of failure. Therefore safety margins are generally not used for serviceability criteria.

Complexity: When limit states design was introduced concern was expressed that it is too complicated for practice. In fact limit states design is as simple or complicated as required to do the job. A key decision in the development of limit states design for buildings was to establish the same load factors and load combinations for all materials in the NBC prior to the development of resistances and resistance factors in the CSA material design standards. The complexity of our structural codes and standards, however, is a serious issue which will be discussed later.

HISTORY OF LIMIT STATES DESIGN

Following some initial studies by Gallileo in 1638 Hooke established the basis of elastic theory (ut tensio sic vis) in 1660. In 1686 Newton published his famous book showing how, from three axioms or laws, we can calculate the motions not only of apples falling to the ground, but also of planets and other bodies of the universe, also the nature of forces acting on them. This theory was eventually applied not only to separate bodies but to the structural behaviour of the bodies themselves (see Appendix A which contains some excerpts from this book). It is clear that Newton was one of the great masters of the thinking process of understanding by modelling. He would have liked today's computers.

In 1741 someone investigated damage to the dome of St. Peter's Cathedral, carried out calculations based on Newton's law of statics, determined the cause of damage to be yielding of the circular tie at the base of the dome, and recommended adding more ties (Institution of Structural Engineers,1980). The report caused a tremendous reaction because, until that time, all structures had been designed and evaluated by experience only.

As an application of his new calculus of variations Euler, in 1757, derived the height of a pole which would buckle under its own weight. His method provided the basis for determining the elastic buckling resistance of compression members.

In 1773, Coulomb derived the critical height of a vertical embankment for a cohesive soil:

[3] $$H_c = 4c.\cos\phi / [\gamma(1-\sin\phi)]$$

where H_c is the critical height and c, ϕ and γ are the cohesion, the angle of internal friction, and the density of the soil. The formula is based on Newton's law of statics, the shear strength of the soil (equal to the sum of its cohesive and frictional resistance), and the assumption that the soil fails by sliding along a plane. This is a version of the limit state formula, Equation 1, useful for civil engineering practice. In 1846, Collin advanced Coulomb's theory to include sloped embankments:

[4] $$H_c = 4\,c.\cos\phi.\sin\beta / \{\gamma[1-\cos(\beta-\phi)]\}$$

where, in addition to the previous notation, β is the slope of the bank. Actually the slip plane is curved (Figure 4) but Equation 4 is a reasonably close solution. In 1857 Rankine established the limit states of active and passive pressure in a cohesionless soil, an application of the theory of plasticity (as compared to simple sliding).

In 1807 Young provided a comprehensive approach to analyzing structures by the application of statics, geometry and the modulus of elasticity, as well as available test data. This was based on scientific investigations since the time of Gallileo. In 1822 Cauchy developed the theory of elasticity using the concepts of stress and strain.

Despite these events, design by experience only continued well into the nineteenth century because, in fact, it worked very well. It worked well because changes in construction materials and procedures were very slow. For example, the first iron bridge built in 1779 (Figure 5), was essentially a replica of a masonry arch bridge, although this was an intelligent decision because cast iron is weak in tension similar to masonry. The industrial revolution, however, changed this approach based on experience only. To handle the rapid changes in materials (in particular steel) and construction techniques, civil engineers developed the methods of Young and Cauchy into practical allowable stress design. An early example is a book on carpentry by Tredgold (1820) in which he showed how to calculate the deflection as well as the

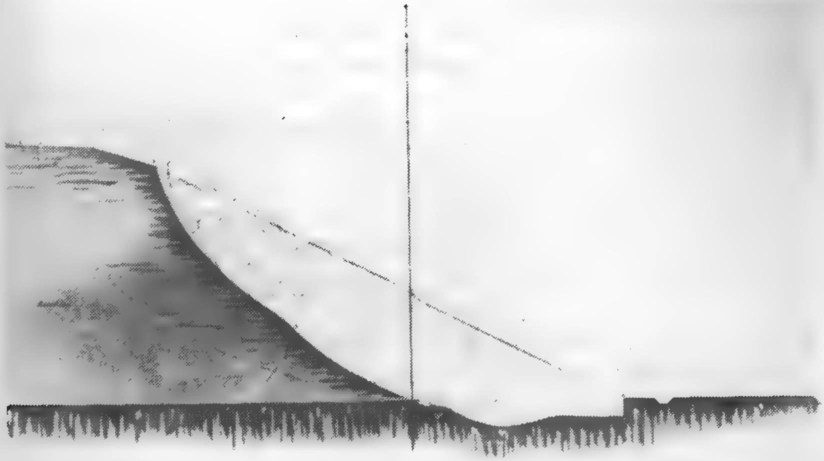


FIG. 4. Embankment - Surface of Failure (Collin 1846)

FIG. 5. The Iron Bridge, Coalbrookdale, England (1779)

stresses (pressures) in wood structures which, along with test data for resistance in tension, compression (parallel and perpendicular to the grain) and flexure, allowed engineers to design wood structures. In this book he recommended a deflection limit of span/480 to prevent damage to plaster ceilings and excessive floor vibration, a criterion that has not changed much over the years. The gradual development of allowable stress

design is reflected in subsequent editions of Tredgold's book up to 1884.

Allowable stress design provided engineers a better understanding of what makes structures fail and, as a result, they were able to successfully design and build structures never dreamed before, long-span bridges and tall buildings. Allowable stress design began to appear in building codes towards the end of the nineteenth century. During this development different modes of failure had to be incorporated such as buckling and fatigue. Allowable stress design was a revolution, because it provided engineers a unified tool for the design and evaluation of any structure. It therefore has features similar to Newton's *'system of the world'*. (See Appendix A.)

In the twentieth century even more rapid changes have taken place. Terzaghi introduced soil mechanics as a tool for practice in 1925. We also know more about how structures behave and elastic theory has been replaced by plastic and other theories, particularly for the ultimate limit states. Equation 4, for example, is based on plastic slip theory, not elastic stresses. In addition, new construction techniques have resulted in lighter structures with longer spans, and serviceability problems such as movements and vibrations have increased as a consequence. Finally safety factors can be determined using risk and reliability methods as well as experience. Limit states design is an evolution of allowable stress design, with the accent changed from elastic theory and material failure to failure of the structure to perform its function.

The first development of limit states design as a unified procedure for all structures was initiated in the USSR in 1954. Outside the USSR the first limit states standard was the 1956 Danish standard for foundations. European countries started developing limit states design in the 1960's, and the International Standards Organization (ISO) adopted it as the basis for international structural design standards in 1973. Limit states design was adopted as an alternative to allowable stress design for buildings in Canada in 1975, the first fully limit states design standard being for steel building structures (CSA S16.1-1974), although the concrete design standard (CSA A23.3), based on ultimate strength design, was close to being a limit states design standard. The conversion from allowable stress to limit states design will be nearly complete in 1995, with limit states codes and standards for most civil engineering applications and materials. The United States is gradually working towards limit states design (called load and resistance factor design) about 5 to 10 years behind Canada.

The United States calls it LRFD (load and resistance factor design), Great Britain calls it limit state design, and we call it limit state<u>s</u> design, the same as ISO. So what's in a label? Actually, everything. Maybe there is a better label than limit states design, but it is much better than the other labels in revealing its essence. Some people call it probabilistic limit states design, but even allowable stress design is *'probabilistic'* in terms of being tested by experience, and could be made *'probabilistic'* if someone wanted to do this.

RECENT ACHIEVEMENTS

I have been happily associated with some of the professors we are honouring in this symposium in the development of limit states design in Canada, as well as others involved with codes and standards, some of whom will be attending the symposium. It has truly been a team effort, in which the sum has been greater than the parts.

Recently, some of us have been involved with the evaluation of existing structures, such as bridges, in particular the new Clause 12 of the CSA S6 standard for bridge design (Buckland et al, 1992). Upgrading an existing structure is much more expensive than ensuring safety in a new structure, consequently criteria which trigger upgrading must be minimized as much as possible for economic reasons. We have developed reduced safety (load and resistance) factors as a function of system behaviour, element behaviour, inspection level, type of traffic and analysis method. These reductions are based on risk, and take into account the likelihood of death or injury given failure, including warning of approaching failure, as well as the probability of failure. A similar approach was recently developed for structural evaluation of existing buildings (in a new Commentary N to Part 4 of the NBC 1995). More research needs to be done, however, in taking into account the fact that old construction systems have performed satisfactorily, despite their inability to satisfy current code criteria.

FUTURE OF LIMIT STATES DESIGN

The problem with the limit states design method is that it is too narrow-minded. It handles only structural and soil mechanics, that is strength, stability, deflection, vibration and local damage. It ignores all other considerations. To illustrate this narrow perspective Table 1 reviews the contents of a typical structural standard to see how well it addresses the problems that occur in practice. The first conclusion of Table 1 is that structural engineers understand ultimate limit states and therefore there are fewer problems (collapses). In fact the space in structural standards devoted to ultimate limit states, currently close to 200 pages, is disproportionate and should be shortened (i.e. simplified).

Table 1. Typical Structural Design Standard

Limit States	Length	Requirements	Problems	Recommendation
Ultimate	100 pages	Many	Few	Simplify
Serviceability	1 page	Few	More	Pull together
Durability	1 sentence	Vague	Most	Improve understanding

The second conclusion of Table 1 is that more should be done about durability. An example for the need of a better approach to durability is the bridge deck/parking garage disintegration problem - a problem that cost billions of dollars every year to repair (a hidden tax). This problem arose from two changes, the introduction of salt as a deicer for roads and the introduction of multistorey parking garages. This was not a new problem in the practice of civil engineering, however. This problem was well known for many years to engineers who design marine structures. Why was it that this problem was not communicated to bridge and building engineers when salt was introduced on roads? The answer is that we, as a profession, did not understand the problem. We did not understand that water gradually penetrates concrete (a porous material) and that the dissolved chlorides in the water depassivate the oxide layer on the steel causing active corrosion of the steel and, as a consequence, either delamination of the concrete cover or fracture of prestressing wires. If we understood this basic failure mechanism, and the environmental conditions which cause it to occur, the communication would have been much better, and more timely action would have been taken to prevent the problem from becoming a major epidemic.

I recall that when the problem first occurred for parking garages in the late 1970's it was blamed on the introduction of limit states design. Actually both allowable stress design and limit states design completely missed this particular failure mechanism. However, it should have been covered by limit states design. Delamination of concrete is local damage which restricts the use of the garage and promotes further corrosion. Fracture of prestressing wires threatens life safety. The legal state-of -the-art answer was *'we did not know it would happen'*. We have since done something about this epidemic, a new CSA standard on parking garages, but what if some other epidemic rears its head?

Limit States Design for Durability

Efforts are being made to prevent or at least minimize these kinds of epidemics in the future. One effort is a CSA guideline on durability in buildings (CSA S478) which is under preparation. The recommendations of the guideline are expressed as follows:

[5] Predicted Service Life > Design Service Life

where design service life of a building component is based on a number of considerations, including life-cycle cost. Hard-to-replace components such as piles or wall connectors obviously require a much longer service life than easy-to-replace components such as paint or shingles. Life-threatening failures must also be checked for safety (Equation 1 with appropriate safety factors), as currently required in structural standards.

The CSA guideline recommends the following methods for predicting service life:

- experience with the same assembly of components (such as a roof, wall, foundation or balcony) in similar environments
- modelling of the deterioration process
- testing

The first and third methods are not new but the second one, modelling, is. Modelling requires a *consideration* of:

(1) the environment adjacent to and within components of an assembly, determined from a *consideration* of the environment outside the building (including the ground), the environment within the building, and the transport and accumulation of environmental agents on the surface and within building assemblies and their components,

(2) the relative movement of adjacent components, determined from the dimensional changes of the components due to stress, temperature, moisture changes and ground movements.

(3) the deterioration or damage mechanisms that occur as a consequence of the environment, as well as the relative movements of connected components,

(4) the limit state (e.g., fracture, damage, gaps, also discolouration is added to the category of serviceability limit states above) defining functional failure of the component (e.g., air barrier, pile, shingle, caulking, connector, protective membrane, visual facade).

This shows that the prediction of service life is obviously much more complicated to model than the structural-mechanics failures embodied in current limit states design. Hence the word used is *'consideration'*, not *'calculation'*. These considerations are currently evaluated by the application of *'building science principles'*, as described in a number of Canadian Building Digests published by NRC. Practical limit states calculations are therefore not yet developed for durability. Aspects of the modelling, however, have been developed, mainly for research purposes. Research is underway at IRC to pull these aspects together into something more nearly like limit states design. One may, for example, determine the moisture content (degree of saturation) of any component in an assembly by computer simulation and apply the following approximate limit states criteria (Bomberg and Shirtliffe, 1992):

Freeze-Thaw: Failure occurs if the moisture content (degree of saturation) is greater than that for which freezing causes fracture

Corrosion/Rotting: The predicted service life is determined when the time of wetness (period of time when the moisture content and temperature are suitable for the process to occur, including weighting functions for its intensity) is sufficient to cause failure

At this stage of development it is better for practitioners to simply be well aware of the basic mechanisms of failure, the conditions under which they occur (moisture with or without contaminants, O_2, CO_2, UV, temperature, etc.), and the basic mechanisms of transport and accumulation of these agents, particularly moisture and air. Guidance on these mechanisms, and the conditions under which they occur, will be contained in the Appendices to the CSA guideline on the durability of buildings. Some models which appear to be useful in describing these mechanisms are:

The Corrosion Cell (Figure 6). This model is useful to indicate the conditions under which corrosion occurs and the type and intensity of corrosion, but it requires explanation (as will be provided in an Appendix of CSA S478).

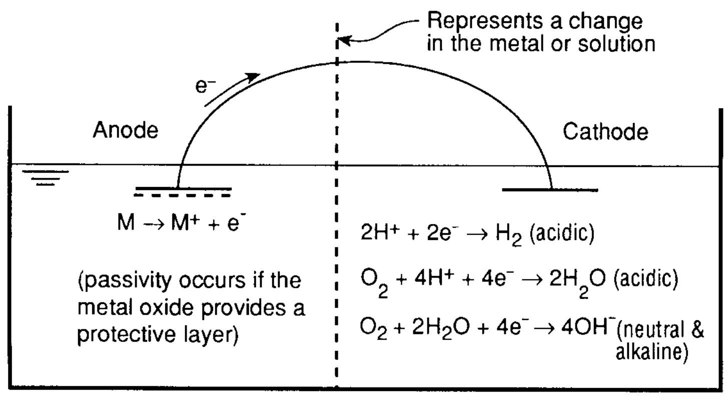


FIG. 6. The Corrosion Cell.

Mechanisms of Rain Penetration. The forces causing rain penetration are kinetic energy of rain drops, capillary suction, gravity and air pressure. The rain screen (Figure 7), a model solution for the prevention of rain penetration, is based primarily on the concept of gravity drainage. A more effective solution is the pressure equalized rain screen, which not only provides gravity drainage but prevents rain penetration through the rain screen due to air pressure.

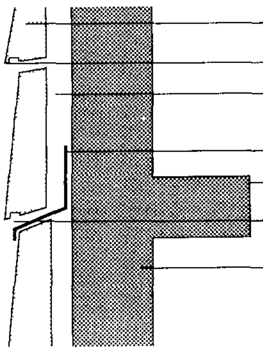


FIG. 7. The Rain Screen

Research is underway towards limit states design for durability, and the features of limit states design will change as a consequence of becoming less narrow-minded. It appears that the development of limit states design for durability is at the same stage as for structural-mechanics in the early nineteenth century, so it will take some time.

Limit States Design for Materials.

We are beginning to have the ability to design and build materials (microstructures). Prescribed molecular arrangements of materials are already being achieved in the laboratory. Also a new scientific endeavor *'nanotechnology'* is devoted to the construction of new materials (essentially a prescribed biological process). These materials should be designed not only to get objects useful for people, but also to avoid failures of various kinds, including deterioration failures. This is another potential application of limit states design concepts.

CONCLUSION

The following paraphrases a comment in a recent magazine article (Peters, 1994): *The best practitioners keep simple models of structures and how they behave in their mind.....so simple that you can mull them over while laying awake at night, driving a car, walking, or keeping yourself occupied during a dull meeting or talk. In addition to this you have to explain your ideas to others, and the only way to do this is by simple models.*

It is therefore important that the models we researchers recommend are suitable for this purpose. Black boxes (formerly handbook tables, now computer printouts) are not acceptable.

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APPENDIX A : EXCERPTS FROM NEWTON, 1686

From '*Preface to the First Edition*"

"Since the ancients (as we are told by Pappus) esteemed the science of mechanics of greatest importance in the investigation of natural things, and the moderns, rejecting substantial forms and occult qualities, have endeavored to subject the phenomena of nature to the laws of mathematics, I have in this treatise cultivated mathematics as far as it relates to philosophy. The ancients considered mechanics in a twofold respect; as rational, which proceeds accurately by demonstration, and practical. To practical mechanics all the manual arts belong, from which mechanics took its name. But as artificers do not work with perfect accuracy, it comes to pass that mechanics is so distinguished from geometry that what is perfectly accurate is called geometrical; what is less so, is called mechanical. However, the errors are not in the art, but in the artificers. He that works with less accuracy is an imperfect mechanic; and if any could work with perfect accuracy, he would be the most perfect mechanic of all, for the description of right lines and circles, upon which geometry is founded, belongs to mechanics. Geometry does not teach us to draw these lines, but requires them to be drawn, for it requires that the learner should first be taught to describe these accurately before he enters upon geometry, then it shows how by these operations problems may be solved. To describe right lines and circles are problems, but not geometrical problems. The solution of these problems is required from mechanics, and by geometry the use of them, when so solved, is shown; and it is the glory of geometry that from those few principles, brought from without, it is able to produce so many things. Therefore geometry is founded in mechanical practice, and is nothing but that part of universal mechanics which accurately proposes and demonstrates the art of measuring. But since the manual arts are chiefly employed in the moving of bodies, it happens that geometry is commonly referred to their magnitude, and mechanics to their motion. In this sense rational mechanics will be the science of motions resulting from any forces whatsoever, and of the forces required to produce any motions, accurately proposed and demonstrated. This part of mechanics, as far as it extended to the five powers which relate to manual arts, was cultivated by the ancients, who considered gravity (it not being a manual power) no otherwise than in moving weights by those powers. But I consider philosophy rather than arts and write not concerning manual but natural powers, and consider chiefly those things which relate to gravity, levity, elastic force, the resistance of fluids, and the like forces, whether attractive or impulsive; and therefore I offer this work as the mathematical principles of philosophy, for the whole burden of philosophy seems to consist in this - from the phenomena of motions to investigate the

forces of nature, and then from these forces to demonstrate the other phenomena; and to this end the general propositions in the first and second Books are directed. In the third Book I give an example of this in the explication of the System of the World; for by the propositions mathematically demonstrated in the former Books, in the third I derive from the celestial phenomena the forces of gravity with which bodies tend to the sun and the several planets. Then from these forces, by other propositions which are also mathematical, I deduce the motions of the planets, the comets, the moon, and the sea. I wish we could derive the rest of the phenomena of Nature by the same kind of reasoning from mechanical principles, for I am induced by many reasons to suspect that they may all depend upon certain forces by which the particles of bodies, by some causes hitherto unknown, are either mutually impelled towards one another, and cohere in regular figures, or are repelled and recede from one another. These forces being unknown, philosophers have hitherto attempted the search of Nature in vain; but I hope the principle here laid down will afford some light either to this or some truer method of philosophy."

Is. Newton

Cambridge, Trinity College, May 8, 1686.

From Book I

"AXIOMS, OR LAWS OF MOTION

Law I

Every body continues in its state of rest, or of uniform motion in a right line, unless it is compelled to change that state by forces impressed upon it.

Projectiles continue in their motions, so far as they are not retarded by the resistance of the air, or impelled downwards by the force of gravity. A top, whose parts by their cohesion are continually drawn aside from rectilinear motions, does not cease its rotation, otherwise than as it is retarded by the air. The greater bodies of the planets and comets, meeting with less resistance in freer spaces, preserve their motions both progressive and circular for a much longer time.

Law II

The change of motion is proportional to the motive force impressed; and is made in the direction of the right line in which that force is impressed.

If any force generates a motion, a double force will generate double the motion, a triple force triple the motion, whether that force be impressed altogether and at once, or gradually and successively. And this motion (being always directed the same way with the generating force), if the body moved before, is added to or subtracted from the former motion, according as they directly conspire with or are directly contrary to each other; or obliquely joined, when they are oblique, so as to produce a new motion compounded from the determination of both.

Law III

To every action there is always opposed an equal reaction: or, the mutual actions of two bodies upon each other are always equal, and directed to contrary parts.

Whatever draws or presses another is as much drawn or pressed by that other. If you press a stone with your finger, the finger is also pressed by the stone. If a horse draws a stone tied to a rope, the horse (if I may so say) will be equally drawn back towards the stone; for the distended rope, by the same endeavor to relax or unbend itself, will draw the horse as much towards the stone as it does the stone towards the horse, and will obstruct the progress of the one as much as it advances that of the other. If a body impinge upon another, and by its force change the motion of the other, that body also (because of the equality of the mutual pressure) will undergo an equal change, in its own motion, towards the contrary part. The changes made by these actions are equal, not in the velocities but in the motions of bodies; that is to say, if the bodies are not hindered by any other impediments. For, because the motions are equally changed, the changes of the velocities made towards contrary parts are inversely proportional to the bodies. This law takes place also in attractions, as will be proved in the next Scholium."

From Book III

"RULE III

The qualities of bodies, which admit neither intensification nor remission of degrees, and which are found to belong to all bodies within the reach of

our experiments, are to be esteemed the universal qualities of all bodies whatsoever.

For since the qualities of bodies are only known to us by experiments, we are to hold for universal all such as universally agree with experiments; and such as are not liable to diminution can never be quite taken away. We are certainly not to relinquish the evidence of experiments for the sake of dreams and vain fictions of our own devising; nor are we to recede from the analogy of Nature, which is wont to be simple, and always consonant to itself. We no other way know the extension of bodies by our senses, nor do these reach it in all bodies; but because we perceive extension in all that are sensible, therefore we ascribe it universally to all others also. That abundance of bodies are hard, we learn by experience; and because the hardness of the whole arises from the hardness of the parts, we therefore justly infer the hardness of the undivided particles not only of the bodies we feel but of all others. That all bodies are impenetrable, we gather not from reason, but from sensation. The bodies which we handle we find impenetrable, and thence conclude impenetrability to be an universal property of all bodies whatsoever. That all bodies are movable, and endowed with certain powers (which we call the inertia) of persevering in their motion, or in their rest, we only infer from the like properties observed in the bodies which we have seen. The extension, hardness, impenetrability, mobility, and inertia of the whole, result from the extension, hardness, impenetrability, mobility, and inertia of the parts; and hence we conclude the least particles of all bodies to be also all extended, and hard and impenetrable, and movable, and endowed with their proper inertia. And this is the foundation of all philosophy."

LEADING CONCRETE DESIGN CODES TO NEW HEIGHTS

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INTRODUCTION

This Symposium honors a group of faculty who collectively, through a broad range of contributions, have brought the University of Alberta to a worldwide leadership position in the field of Structural Engineering. It is a fitting place to discuss the characteristics of leadership in the formulation of codes of practice and particularly building codes for structural concrete.

In our North American democracies, the tradition of developing a clear consensus in support of a proposed change is the rule in formulating concrete design codes. Such a consensus means overwhelming agreement of a committee of strong-willed persons representing a broad myriad of conflicting interests after consideration and judgement of the merit of every dissenting opinion. In order to obtain such a consensus, code recommendations must not only be technically sound, but they must be practical, workable, and necessary. Above all, they must be patiently and painstakingly presented, defended, revised and re-presented to the committee. Those who shepherd the proposals which ultimately become part of the standards which guide practice and protect the citizenry must be effective leaders.

The New Lexicon Webster's Dictionary defines a Leader as someone who acts as a guide -- a directing head or chief. In its definition of the verb, *to lead*, Webster's has several expressions germane to the present discussion:

- *to show someone the way to go by accompanying him (or her)*
- *to persuade to do or believe something*
- *to bring about a specified result*
- *to direct, supervise the actions, policies, workings of a group*
- *to hold first place in*

Max DePree, in his excellent book "***Leadership is an Art***," says "*Leaders, in a special way, are liable for what happens in the future, rather than what is happening day to day.*" This is certainly true and directly applicable to those who formulate, propose, defend, sell and explain North American structural concrete code provisions. Their recommendations become, in fact, prescriptions for future practice which guide

the activities of designers from Atlantic to Pacific and which vitally affect the design and construction of myriad buildings, bridges and special structures. These recommendations vitally affect the safety of hundreds of millions who use and are sheltered by these structures, as well as the fortunes of millions of owners who implicitly rely on the wisdom of the designers shaped by acceptable codes of practice to make their structures serviceable and durable. Enshrinement in a building code is the practical path for implementation of technical ideas and research results. In order to achieve this status, the recommendation must not only be technically sound, practically obtainable, and clearly important for safety or serviceability, but it also must be clearly articulated and defended by a skillful and tenacious proponent. These proponents must have thick skins, substantial technical knowledge about the proposal, and the ability to modify their proposals to win over opponents while maintaining the true core important to the original adherents. It is a tortuous process, necessary for developing the compromises so important when forging legislation in the forums of true democracies. The proponent must be a leader in the broadest term. The leader must live up to Max DePree's edict, "***Leaders don't inflict pain; they bear pain.***"

In this essay we will trace the development of a particular leader in the arena of structural concrete design codes and hopefully learn some lessons which others who aspire to such leadership might be wise to emulate.

LEADER -- SOMEONE WHO ACTS AS A GUIDE

An essential requirement of a successful guide is that he or she knows the route to be followed. Hopefully, the guide should have been over the route previously but in truly pioneering situations that is impossible. As a minimum the guide should have listened carefully to others who may have blazed the trail, possess good instincts and feel for direction, and have successfully completed other hazardous passages. In today's symposium we are honoring several who have made important contributions to codes and standards. Without wishing to slight any of the others, I want to focus in particular on the contributions of one engineer who has become one of the most significant contributors to structural concrete design code development in North America, Professor Emeritus James G. MacGregor.

Jim MacGregor was educated to be a leader and showed distinction as an undergraduate here at Alberta. As a graduate student at Illinois Jim worked closely with two important leaders in concrete design code development -- Chester Siess and Mete Sozen. When I first became acquainted with Jim, he was a research associate of Professor Siess, handling much of Chet's correspondence dealing with recommendations for ACI 318 building Code change proposals. Under Chet Siess' tutelage, Jim learned the secrets which make the process effective -- technical correctness, precision and clarity of language, transparency and simplicity of use, illustrative trials with practical examples, willingness to understand and appreciate the views of others, and persistence in striving for an acceptable and honorable compromise.

Jim also learned that the most effective spokesman on a blue ribbon code committee is one who is recognized as a leader in the context of holding first place in his own field. Whether research, design, construction or technical association activity, the effective leader must be a recognized prophet or outstanding doer in his or her own field of activity. On his return to the faculty at Alberta, he immersed himself in a broad range of activities which were to lead to international recognition as an expert in structural concrete behavior and in quantifying structural safety.

LEADERSHIP - SHEAR AND DIAGONAL TENSION

In his graduate research at Illinois, MacGregor learned the importance of structural behavior as the key to development of design models. His deep involvement with the shear behavior of reinforced concrete and prestressed concrete members at Illinois served as a springboard to his earliest international recognition as a leader. Interestingly, this recognition, the 1968 ASCE State of the Art Award for collaborative work with the Joint ACI-ASCE Committee on Shear and Diagonal Tension, was for an excellent synthesis summary of previous experimental and analytical work on shear which led to a new design model. As with so many of Jim MacGregor's distinguished contributions, it highlighted his ability to synthesize -- to find the similarities between his problem and the material he found in theses and dissertations, reports, published papers, committee documents and text books. While a capable laboratory investigator, throughout his career MacGregor relied much more on the assembly and synthesis of the laboratory results of other than on a preoccupation with his own results. The early 1968 ASCE Award was quickly followed by another 1974 ASCE State of the Art Award, again for a committee report on shear. The Code formulation proposal reflecting these studies earned for Jim the 1976 ASCE R.C. Reese Prize for shear design provisions.

As a Member and then Chairman of the Joint ACI-ASCE Committee on Shear-Diagonal Tension, as Chairman of ACI 318 Subcommittee E responsible for shear and torsion design provisions for the ACI Building Code, as Chair of the Canadian Standards Association Task Committee on Reinforced Concrete Design and as Member of the Joint Committee on Reinforced Concrete Design of the National Building Code of Canada, MacGregor led a series of almost continuous development of building code provisions (shown in Table 1) which provided design models for one- and two-way shear, integrated torsion with shear and flexure, and developed strut-and-tie models for D-Discontinuity regions. While these developments encompassed his own tests, the latter were a small fraction of the overall base for the recommendations. He led a great process of synthesis, merging extremely scattered test results and fragmented design theories into comprehensive Code provisions. His contributions in this area were also recognized by his demand worldwide as a lecturer on shear design (See Figure 1).

Table 1

MacGregor Contributions to

Shear and Torsion

- Performed definitive tests on shear-diagonal tension capacity of prestressed concrete members
- Synthesized shear test results into a framework which emphasized a/d or M/Vd effect
- Established one-way and two-way shear approach
- Incorporated axial load effects in shear provisions
- Incorporated torsion effects in shear provisions
- Led adoption of variable angle truss model for combined N, V and T
- Developed deep beam design procedures
- Proponent of strut-and-tie models for consistent, highly transparent detailing of discontinuity regions

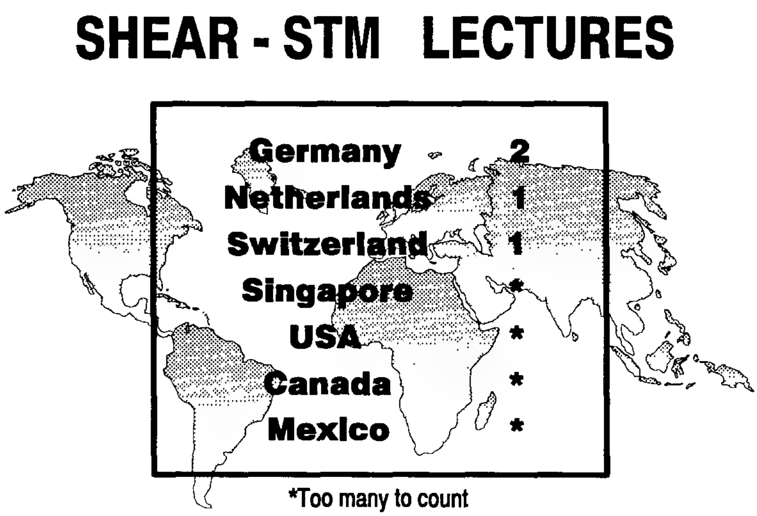


Figure 1

LEADERSHIP - SLENDER COLUMN STABILITY

His interest in slender columns began while working as a Research Associate at Illinois where he helped review proposals and progress reports for the Reinforced Concrete Research Council project on Slender Columns at The University of Texas. His insightful advice to the project investigators and his keen following of their results made him a major participant in the later formulation of design provisions. Working with Joint ACI-ASCE Committee on Reinforced Concrete Columns, MacGregor carried out major computer studies and again brought his talents for synthesis and his ability to formulate a transparent, behaviorally correct model to bear on the slender column design procedures. He was a principal author of the "Moment Magnifier Method" for slender column design which in 1971 replaced the behaviorally incorrect "Load Reduction Factor" method which had historically been the accepted procedure in the ACI Building Code. The moment magnifier proposal (δ) resulted in a calculation procedure which permitted the designer to closely track the actual load path of a slender column (see Figure 2) and hence predict an ultimate load, P_4, which was very close to the actual axial load capacity of tested columns, P_3. This load often represented a significant decrease from its nominal axial load capacity, P_1, predicted by a first order analysis. In addition, the δ procedure gave the increase in moment ($P\Delta$) which would occur. The latter is extremely important in laterally loaded sway frames as the restraining beams must be designed to resist this increased moment or premature failure would occur. The earlier Load Reduction procedure of the ACI Building Code resulted in very unconservative axial loads, P_2, and could not give this vital information about necessary restraint moment increases. MacGregor's jointly authored paper on the slenderness design provision was so well thought of that two award groups each insisted on giving it their prize - it received both the ACI 1972 R.C. Reese Medal and the ACI

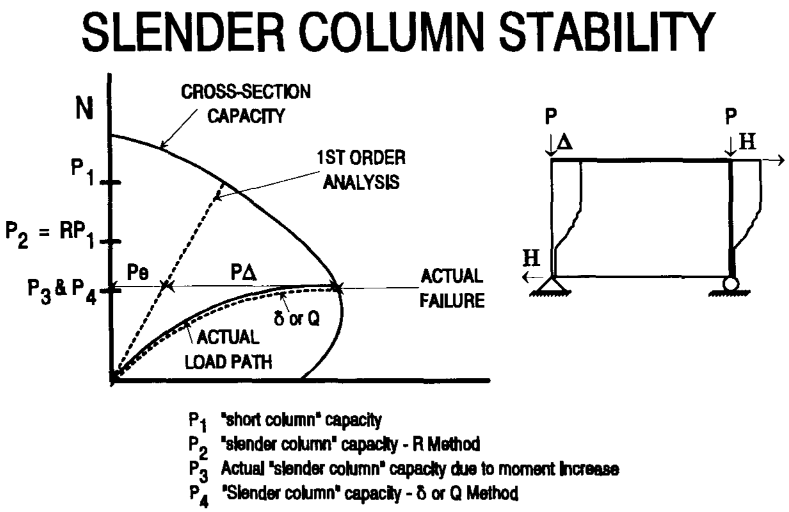


Figure 2

1972 Wason Medal for the Most Meritorious Paper. (Action was taken by the ACI Board to prevent future papers from being doubly honored, but again MacGregor was a leader.)

However, as Max DePree indicates, "*Leaders are obligated to provide and*A true leader is never satisfied with the status quo, even if it is their own status quo. *maintain momentum.*" As shown in Table 2, Jim MacGregor continued to provide leadership in improving the approximate column slenderness design procedures and in refining methods of calculation for the more "exact" second analysis procedures. His Q procedure, influenced by his Alberta colleague, Peter Adams, and his second order analysis procedures are more suited for computer calculations than the δ method. Throughout his career, Jim stayed abreast of continuing office practice changes and recognized the growing importance of computer-aided design. His later contributions on stability received the ASCE R.C. Reese Prize in 1979 and adoption in the Building Codes. He has lectured worldwide on concrete stability procedures (see Figure 3).

LEADERSHIP - STRUCTURAL SAFETY

The overriding purpose of building codes are to protect the public safety. Throughout the parade of centuries, men have been conscious of the need.

In about 1800 B.C., the Emperor Hammurabi in ancient Babylonia codified the rules and regulations of his kingdom. The entire system of laws were written in cuneiform on huge pillars which were copied to ensure that all governors and judges used a uniform code. Included within this code of Hammurabi was the first known

Table 2

MacGregor Contributions to Slender Column Stability

- **Conclusively showed R Method of ACI 318-63 was unsafe and misleading**
- **Introduced Moment Magnifier Approximate Procedure which was safe and emphasized secondary moment increase (ACI 318-71)**
- **Suggested refinements in Moment Magnifier**
- **Introduced Stability Index (Q) Procedure as alternate to Moment Magnifier which was better suited to computer use.**
- **Formulated rules for more rigorous second order analysis (ACI 318-95)**

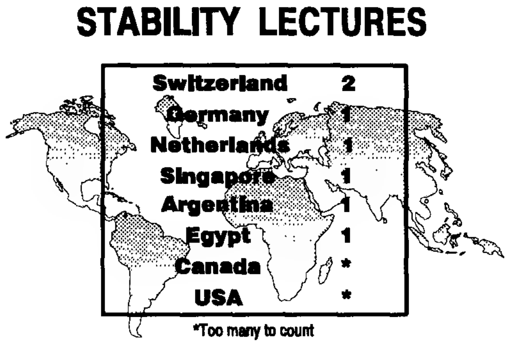


Figure 3

"uniform building code." In recent years many have advocated the introduction of the "performance code" as contrasted to the "specification code." A specification type code imposes limiting values on the design of a product or the constituents of a material: how many 2×4 studs in a wall, for example, or how much manganese in a melt of steel. A performance type code imposes limiting values on how the product or material will behave under a given set of conditions: the minimum acceptable sound transmission characteristics of the wall, for example, or the minimum acceptable ductility of the steel. Specification type codes tend to inhibit innovation while performance type codes set goals for the designers to achieve, although providing little guidance for their attainment. Code committees endlessly debate how much performance orientation is practical. Hammurabi was clearly a "performance man." His code stated:

- If a builder build a house for a man and complete it, (that man) shall give him two shekels of silver per SAR of house as his wage.
- If a builder build a house for a man and does not make its construction firm, and the house which he has built collapses and causes the death of the owner of the house, that builder shall be put to death.
- If it cause the death of a son of the owner of the house, they shall put to death a son of that builder.
- If it cause the death of a slave of the owner of the house, he shall give to the owner of the house a slave of equal value.
- If it destroy property, he shall restore whatever it destroyed, and because he did not make the house which he built firm and it collapsed, he shall rebuild the house which collapsed from his own property (i.e., at his own expense).
- If a builder build a house for a man and does not make its construction meet the requirement and a wall fall in, that builder shall strengthen that wall at his own expense.

If nothing else, Hammurabi's Code must have ensured that builders' sons took a lively interest in their fathers' quality assurance programs.

The obvious elements of Hammurabi's Building Code are:

- clarity and ease of interpretation
- appreciation of benefits to society
- certain and dramatic enforcement

These elements are essential today. Jim's contributions to structural safety clearly encompassed the first two of these elements.

In the period 1975-77, Jim undertook a travelling lectureship for the Canadian Society for Civil Engineering on the subject of "*Safety and Limit States Design for Reinforced Concrete*." He again synthesized a great deal of previous studies and research in these fields, giving an insightful, practical and inspiring approach for a modern structural safety approach for structural concrete.

As we move towards improvement and refinements in design and in materials control procedures, more and more we are utilizing probability concepts. Structural safety, in a somewhat oversimplified view, as shown in Figure 4, may be determined by considering as normally distributed random variables both the loads which come onto a structure and the resistance of the various elements of the structure to those loads. As shown in Figure 5, the chance of a certain load U coming on the structure can be reasonably predicted if there is good knowledge of the likelihood of various loadings that the structure might be subjected to, as well as good means of analysis of the structure. Similarly, the probable resistance R of the structural elements can be predicted, based on the knowledge of their probable dimensions, proportions, material qualities, and the construction accuracy (see Figure 6). The chance or probability that the load U might exceed the resistance R and cause a failure can be determined from Figure 4 as the shaded area where these curves overlap. The smaller this overlap area is, the smaller the chance of failure. Safety factors can be applied through use of load factors and resistance factors, or of other methods to space these curves apart in order to reduce the probability of failure to some acceptable level. This conceptual representation of safety is basic and leads to a clearer understanding of what we do when we try to set safety factors.

W.W. Lowrance, in his excellent book "*Of Acceptable Risk*," emphasizes the distinction between measuring risk and judging safety.

> "MEASURING RISK — Measuring the probability and severity of harm — is an empirical, scientific activity."

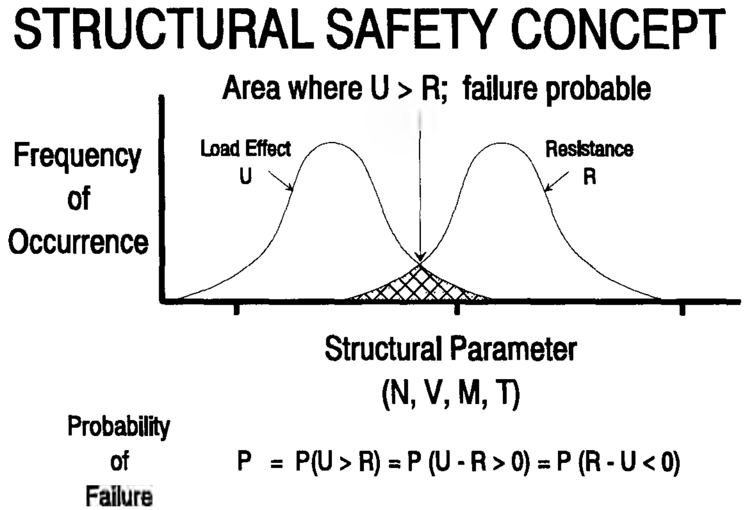


Figure 4

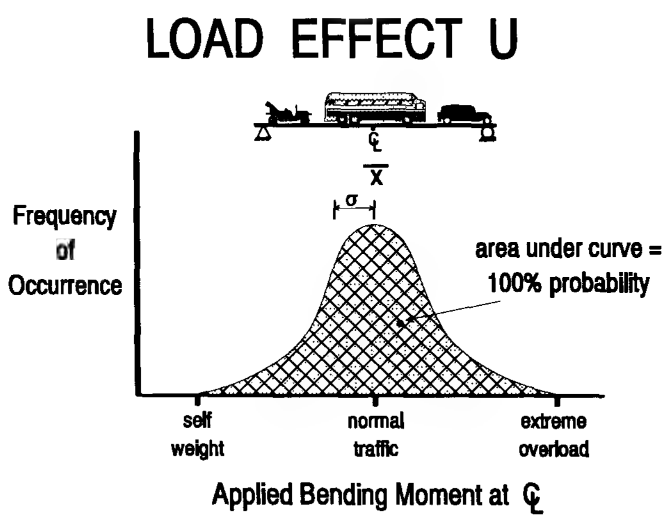


Figure 5

RESISTANCE R

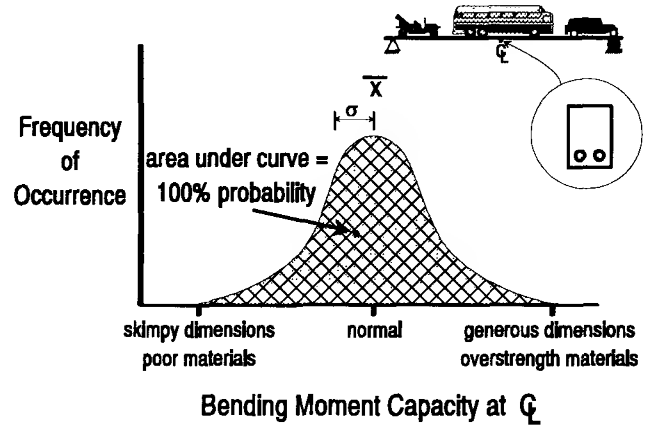


Figure 6

On the other hand,

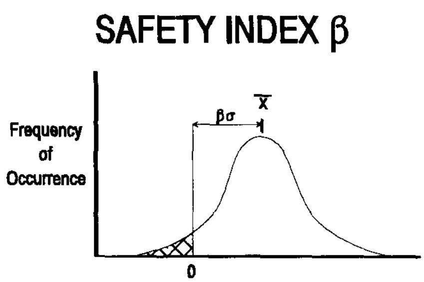
> "JUDGING SAFETY — Judging the acceptability of risks — is a normative, political activity."

While the difference between the two seems obvious when stated this way, it is in fact not so obvious. Many who have led in the development of probablistic methods for measuring risk are impatient with the slow acceptance and skepticism sometimes expressed by those whose responsibility is judging safety. Conversely, many of those who are basically responsible for the judgement of safety have ignored the substantial benefits available from an active dialogue with those who measure risks. In modern times, there is no higher responsibility for the structural engineer than his or her responsibility to safeguard human life. Many engineers confuse the scientific role for which they have received training in depth with the political role for which they are sometimes ill-prepared. Many engineers object to the less technically trained lay representatives of the public participating in building code development. The selection of the appropriate level of safety involves far more human judgment and is much more complex than the solution of systems of differential equations.

Modern approaches to determination of structural safety factors are based on comprehensive probablistic studies. Two different approaches have been suggested: risk analysis and calibration. Both are used or cited for determining how much overlap should be allowed between load effects and resistance. However the choice is made, it can have great social and economic consequences on life safety and on utilization of materials. It is a serious question that all in the profession should be aware of and discuss. In spite of rivalries between construction materials, it is fundamentally a broad

question that should apply to all materials with proper consideration of the variability and experience record of each material, so that the general public will be equally protected in all modern forms of construction.

MacGregor's contributions were not only in gaining general acceptance on the philosophical level, but very importantly were on the quantification, particularly of R. Working with Mirza and others at Alberta, he examined a broad series of test and professional reports to determine the probable levels of variations in all the factors affecting resistance (Figure 6). These considered not only structural theories and measured material strength variations, but also tolerances and field measurements from as-built structures. Working with a blue-ribbon panel in 1979 at the National Bureau of Standards, he investigated the Safety Index, β, inherent in then current building codes. β, shown in Figure 7, is a measure which represents in an indirect way the probability of failure for given statistical distributions of load and resistance. MacGregor led the effort to determine then current β levels implicit in the ACI Building Code. Such knowledge is essential for use of an alternate method for judging safety which is termed calibration. In calibration you go through probablistic type studies to identify the load and resistance curves, but then space the curves apart a distance which will provide a probability of failure which is similar to present safety factors, or which reflects minor changes in current factors. One might wonder why such elaborate studies should be run if the answers are to be adjusted to give the current safety factors. The advantage of the calibration procedure is that such a thorough examination of codes can highlight otherwise overlooked anomalies. This technique shows the great apparent disparity in the relative probabilities of failure between gravity, wind, and earthquake loading. With the memories of the Northridge earthquake disaster fresh, we should be carefully examining the high apparent risks that we are accepting in major cities where such natural disasters threaten. Even within individual material codes, such as the ACI Building Code, anomalies are highlighted by the calibration process. MacGregor's studies showed that in concrete the resistance factor for shear is too high relative to the resistance factor for flexure. Overall structural safety could be improved by making simple changes which would eliminate these inequities.



$P\,(R - U)$

$P = P\,(R - U < 0)$

Figure 7

Table 3

MacGregor Contributions to Structural Safety

- Articulation of Limit State Philosophy
- Definition of Resistance Statistics for Reinforced and Prestressed Concrete Members under N, V, M, T Combinations
- Determination of β under Existing Building Codes
- Harmonization of P_f for Structural Concrete with Structural Steel, Masonry and Timber
- Selection of Target β values by Calibration
- Proposed Common Load Factor Combinations for All Materials (ASCE 7 Adopted)
- Proposed Resistance Factors for Structural Concrete to Yield Target β Values (ACI 318 Adopted for Mixed Construction)

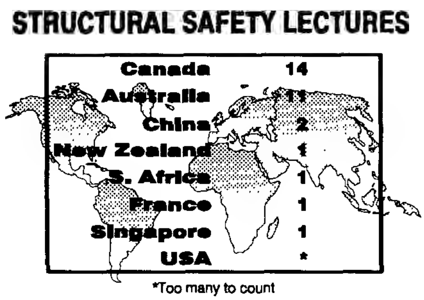


Figure 8

These landmark studies provided the basis for the recent LRFD code adopted by AISC, the basic load factors adopted by the ASCE 7 loads standards, and the newly adopted alternate load factors and resistance factors to be used in the 1995 ACI Building Code for design of composite steel-concrete structures. The overall limit states-structural safety philosophy has been implemented in the Canadian codes. MacGregor and his co-workers at the National Bureau of Standards program received one of ASCE's highest honors, the 1983 Norman Medal for this pioneering effort. His contributions to structural safety, shown in Table 3, led to requests for lectures around the world, as shown in Figure 8. There is no one who has expressed the safety theory for concrete structures so clearly.

LEGACY OF A LEADER

DePree says "*Leaders should leave behind them assets and a legacy.*" Throughout his career Jim MacGregor has given the structural engineering profession a series of outstanding assets. His writings on a wide range of subjects (see Figure 9) have graced the pages of major North American and international journals (see Figure 10). Their quality has been vividly attested to by the recognition of his peers in Canada (Table 4) and throughout North America (Table 5). Not only are his writings in formal publications important, but serious students can learn from the minutes and documents of the ACI Building Code that Jim MacGregor has been a consistent source of momentum for developing sound provisions from a variety of new and old ideas.

Certainly, in terms of a legacy, one must mention the capstone that Jim provided in his two editions of his Prentice-Hall text ***Reinforced Concrete: Mechanics and Design***. Its comprehensiveness combined with its originality, established it as a major text in the field and an important part of the legacy of a great among the leaders of structural concrete design practices. Those of us who are his friends look forward to many more editions to pass on the thoughts of this gentle giant from his new home on the BC coast.

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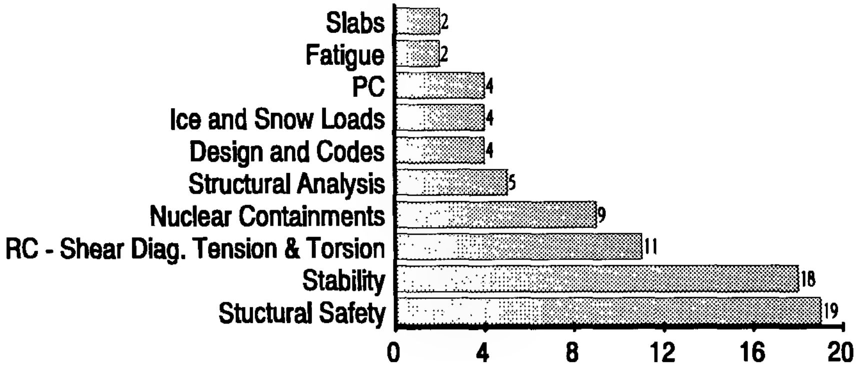

Figure 9

PUBLICATION LOCATIONS

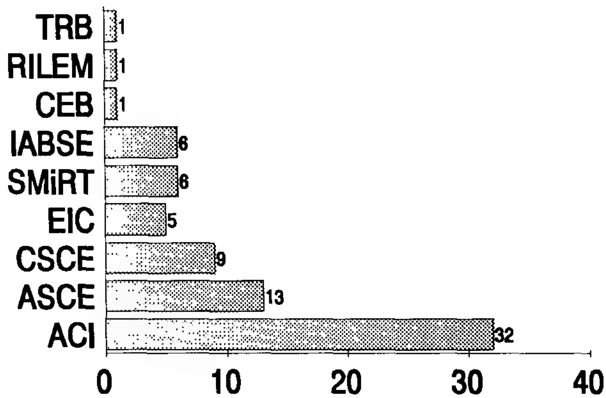

Figure 10

Table 4

RECOGNIZED BY HIS PEERS AS A LEADER

Alberta	Association of Professional Engineers - Gold Medal 1956 Alberta Achievement Award - 1972 Professional Engineers Association - Centennial Award 1986 University of Alberta Kaplin Research Prize - 1990
Canada	Canadian Government Dinner w/ Queen Elizabeth - 1975 Canadian Society for Civil Engineering - 1976 Lecture Tour Royal Society of Canada - Fellow 1982 Canadian Society for Civil Engineering - Fellow 1983 Canadian Academy of Engineering - Fellow 1989 Canadian Standards Association - Award of Merit 1991

Table 5

RECOGNIZED BY HIS PEERS AS A LEADER

American Society of Civil Engineers	American Concrete Institute
State of Art Award - 1968 - (Shear)	R.C. Reese Medal - 1972 - (Stability)
State of Art Award - 1974 - (Shear)	Wason Medal for Best Paper - 1972 - (Stability)
R.C. Reese Prize - 1976 - (Shear)	Fellow - 1973
Fellow - 1978	Del Bloem Award - 1974 - (Shear)
R.C. Reese Prize - 1979 - (Stability)	Kelly Award - 1986 - (Teaching)
Can-Am Engineering Amity Award - 1979	R.C. Reese Award - 1987 - (Deep Beams)
Norman Medal - 1983 - (Structural Safety)	P.M. Ferguson Lecturer - 1988 - (Building Codes)
RCRC Boase Award - 1985 - (Building Codes)	Director - 1972-75
	Vice President - 1990-92
	President - 1992-93

CONCLUSION

In our little world of the American Concrete Institute, which I have had the privilege of sharing with Jim, I have been able to observe him as a technical specialist and a co-worker on the Column Committee and the Building Code Committee. I have served with him on the ACI Technical Activities Committee where a broad range of technical matters are considered. I have observed him rise through the Board of Direction, through the Vice Presidencies, and become President of the Institute. Every step of the way he provided service and inspiration to others. I am a great believer in the consensus process and I believe that I can detect consensus formation as well as most men. It is clear to me that in the international community of those who develop structural concrete building code provisions there is a clear consensus which Webster could accurately add as a new definition of the verb *to lead* in his dictionary:

"To lead in the field of structural concrete building codes is to be a Jim MacGregor."

-- Consensus --

COMPOSITE COLUMNS
ESTRANGED RELATIVE OF STEEL AND CONCRETE STRUCTURES

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ABSTRACT

Steel-concrete composite columns are both reinforced concrete members as well as structural steel members. Instead of serving as an interface between structural steel and reinforced concrete design methodologies, they have had scant attention from either ACI318 or AISC. Designs based on the AISC methodology would not produce the same member sizes as designs for the same conditions if the ACI318 rules were applied. A brief review of the evolution of distinct methodologies, one for structural steel and the other for reinforced concrete, follows. A discussion of current design standards, based primarily on a limit state of strength, identifies major areas of discrepancy in design results. However, the strength design procedures do provide a common basis from which composite columns can serve to unify structural steel and reinforced concrete design standards.

THE FAMILY OF CONSTRUCTION MATERIALS

Construction materials historically have changed with their availability and with the builder's understanding of their use. From woven thatch to simple wood poles, from timber to stone, the progress of civilization is reflected by the materials of construction. Both concrete and iron-based materials have been known and used for centuries, but the modern forms of concrete and steel had their birth almost simultaneously early in the nineteenth century.

The first patent for portland cement was granted in 1824, and the first rotary kilns for producing portland cement in North America were installed in the 1850's (MacGregor, 1988). The puddling process and the use of coke instead of wood or coal for iron production were initiated before 1800. Cort's process of using forced air led to the production of wrought steel elements that were large enough for structural components. The Bessemer process, which dates from 1856 (US Steel 1971) made possible the production of a volume of steel large enough to feed the voracious new

railroad industry. Some failures of cast iron structures had delayed acceptance of its cousin steel, but by 1890, ductile wrought and rolled steel shapes had replaced cast iron for most applications in metal structures. Cast iron pipe filled with concrete remained in use for columns.

Concrete was used initially as artificial or fabricated rock, and wrought steel was used initially as a substitute for wood. The encasement of steel shapes with concrete for insulation led to the obvious conclusion that, structurally, there was no need for the steel in flexural compression. Concrete members reinforced with steel shapes represented the original form of reinforced concrete. Bar reinforcement for concrete members developed in a somewhat torturous manner. Numerous patents were awarded for specialized bar shapes and applications. Prior to World War I, many concrete structures were built by contractors who were educated by the "try it and test it" experience, and who obtained patents with priority rights to control and profit from the use of those systems which worked (Hool 1913). The proof-load test was an accepted standard for construction approval.

Structural steel frames were applied to bridges and to high rise buildings before 1910. Reinforced concrete, recognized for its fire resistant nature, also found use in highrise buildings. The family of structural concrete and the family of structural steel, nurtured under the same roof, remained essentially together until after World War I.

FAMILIES GROW APART WITH MATURITY

Many of the mills that produced material for structural shapes also made concrete reinforcing bars. Reinforcing bars required less precise controls and handling and represented the least expensive finished product from the mills. With bar reinforcement, the use of structural shapes for concrete reinforcement diminished rapidly, all but ending after the third decade of the century. Many fabricators of structural steel also cut and bent reinforcing bars. However, few of the same tools or labor skills applied or were used for both shapes and bars. The sources and promoters of income for the family of structural steel became quite distinct from those of the family of reinforced concrete. Quite apart from the producers of steel, the family of reinforced concrete was supported by producers of cement, aggregate, and equipment for mixing, placing, and forming concrete.

Determinate Versus Indeterminate

Further separation between the family of structural steel and the family of reinforced concrete was a natural consequence of the form and function of frame components constructed by each family. The well articulated, virtually hinged connections of structural steel frames made the use of equations of equilibrium completely adequate for the determination of internal forces on frame components. A high level of confidence could be expected that analytic estimates of internal forces corresponded with actual shears, moments, and thrusts in steel structures. Allowable stress values for

estimated conditions in determinate frames served as the basis for proportioning reliable structural steel systems. In contrast, cast-in-place concrete frames had rigid joints and no articulation of connections at joints. Without convenient tools for using displacement compatibility relationships for estimates of internal forces on concrete frame components, designers of concrete structures relied on empirical systems from trial and experience. Some designers were recognized for success with concrete systems and marketed their professional services on that basis. Structural redundancy in concrete structures was known but not highly regarded as a virtue.

Stress Design Versus Strength Design

Not only were the demands of structural analysis more complex for continuous concrete structures than were analytic challenges for determinate steel systems, but also the users of concrete knew that strains were not proportional to actual stresses as they had to assume for elastic analysis of internal forces. Designers of concrete systems did develop increasing confidence that estimates of cross section strength of concrete elements could be predicted more easily and more accurately than estimates of stresses. In 1956 the first Strength Design rules appeared in an appendix of the ACI318-56 Building Code (ACI318-56). Strength Design appeared as an acceptable alternate for the next decade, until allowable stress design appears today in the ACI Code only as an appended "alternate method". The AISC family of structural steel continued to use Allowable Stress Design.

Slender Columns Versus Stocky Beam-columns

Performance of slender steel columns had little in common with performance of concrete columns. Virtual hinges at beam-to-column joints made steel columns behave as pin-ended struts between supported beams. Slenderness was virtually the only criterion for proportioning safe steel columns. Extensive research examined very carefully the effects of slenderness, residual stresses, lateral and torsional buckling, and local buckling of steel components in order to fine tune strength-slenderness functions for allowable compression stress on axially loaded steel columns. An allowable stress limit for the sum of axial and flexural stress was adequate to accommodate the infrequent occurrence of steel beam columns.

In contrast, concrete columns were not likely to be slender, and local buckling of column components was unthinkable. Crude relationships were adequate for reducing axial load limits for the rare concrete columns that were taller than 10 times their thickness. When analytic procedures were developed for estimating indeterminate joint moments, concrete columns had to be designed always as beam-columns. The very existence of axially loaded concrete columns was denied by ACI318, as attention focused on reliable estimates of section strength. Section strength became the only criterion for most concrete columns.

A family of structural steel and a family of reinforced concrete became easily identified. With separate revenue sources as sponsors, with structural behavior as

determinate systems and not indeterminate systems, with allowable stress instead of limit strength as a basis for design, and with unrelated procedures for designing columns, the two families had little need for one another.

BOTH FAMILIES NEED CONCRETE SLABS

No material has better economy, strength, and durability as a floor surface than does concrete. Before mid-century, reinforced concrete slabs were used as a floor surface designed as if supported by steel joists, stringers, or beams. Observations of performance suggested that composite flexural actions occurred, since slabs participated as part of resistance to flexural compression forces. Laboratory studies helped define the flexural mechanics of composite concrete slabs acting with structural steel shapes. Requirements for shear connectors to insure reliable unified performance of the concrete slab on steel beams were developed.

A comprehensive report for recommended practice with steel-concrete composite beams appeared in 1960 (ACI-ASCE 1960). The family of structural steel adapted the standard into the AISC Specifications (AISC1964) as a part of the rules for steel-supported floor systems. Composite steel-concrete flexural members were designed with rules from the AISC family of steel. Proportioning of the slab itself with provisions for flexural reinforcement of the slab were not mentioned in the AISC rules, as the family of reinforced concrete should and did provide guidance for concrete slabs. The ACI318 Building Code was used for proportioning the slabs, but the Code in its own turn ignored regulations for the steel supports.

Steel-concrete composite beams were respected by both families, but only marginally nurtured by either. Unpleasantness happened. The traditional concept that steel stringers and beams had hinged connectors coupled with conceptual design of concrete slabs only for strength and performance between and perpendicular to supporting beams, did produce safe floors, but floors with unpleasant cracks which outlined the girders which supported "hinged" beams.

COMPOSITE COLUMNS

Since columns were recognized as compression members, and concrete was designed for its compression reliability, design regulation of composite columns was relegated to the domain of the concrete family. Whether or not they were attached to beams with virtually hinged shear connectors, composite columns were to be designed as eccentrically loaded compression members consistent with strength design procedures for proper concrete columns. Composite steel-concrete columns in the form of concrete encased steel shapes or as concrete-filled steel pipes, quite common in 1900, appeared less frequently as the century progressed.

The concrete-insulated steel shape composite column looked and behaved

virtually the same as other tied concrete columns. The insulation of steel shapes with concrete was used for institutional and industrial applications, although the concrete was assigned very little (or none) of the load-carrying burden of such columns. The insulation of steel columns can be accomplished with far less expensive methods than encasement in concrete. The use of concrete encased steel shapes as composite columns was all but discontinued.

Concrete-filled steel pipes looked and performed more like structural steel columns than reinforced concrete columns. These columns frequently exhibited slenderness ratios for which ACI318 reduced allowable axial strength from an already too small axial load capacity that permitted less stress on steel than that allowed by AISC rules. Concrete-filled steel tubes were acknowledged to be very tough, and they found use in such areas as loading docks. Proprietary composite filled tube columns (Lally columns) could be designed apart from ACI rules relying instead on test data from the proprietor. Even Lally columns all but disappeared from use after 1950.

Concrete-filled steel tube columns in Japan were observed to have survived the "Great Kanto Earthquake" in 1923, and Japanese use of concrete- filled tube columns continues today (Wakabayashi, 1978). North Americans were not familiar with Japanese practices before the end of World War II. The inherent toughness, optimal efficiency of steel with concrete, and absence of column formwork encouraged widespread use of concrete-filled tube composite columns only in labor-intensive economies. Such columns have become rare in North America.

The rather sparse use of composite columns during the middle decades of the century did not encourage efforts to improve design procedures that would improve the degree of reliability or consequent efficiency of composite columns. Without a visible identity either as steel columns or concrete columns, composite columns remain an estranged part of the construction materials family. Periodically, efficient new applications for composite columns are discovered, and the estranged relative is put to work with one family or the other.

Rationalization of Slenderness For Both Steel and Concrete Families

Column design became more sophisticated after 1950 with the rationalization of slenderness effects. The use of a beam-column formula reduction for column stress due to eccentric compression had been a part of bridge design standards (AASHO 1957), but its use for structural steel had been neglected by AISC specifications and by the ACI Building Code, since there were very few beam-columns used in structural steel frames, and there were very few slender concrete columns.

Advantages of continuous, rigid framing, and the growth of plastic design as well as moment distribution for pre-computer estimates of forces in indeterminate structures, may have served as strong motivations for the development of beam-column design procedures applied to slender steel columns. A moment magnification procedure appeared in the AISC Manual in 1964. The same formulation for moment magnification in design for slender concrete columns appeared in ACI318 in 1971 (ACI318-71). Basic

concepts for both systems are identical. Applications differ.

Strength Design Offers Grounds for Family Reunion

ACI318 has recognized strength design since 1963, and it serves today as the primary method for reinforced concrete design. Strength design procedures were formulated and published by the structural steel family in Canada during the decade of the seventies, and in the US it appeared in an AISC Manual as Load and Resistance Factor Design in 1986 (LRFD, 1986).

Load factors initially used for ACI318 were adapted largely from judgement and the experience of ACI318 committee members as well as from proposals then in effect in European design standards for concrete structures. The initial load factors have been adjusted somewhat since the initial values were set. After a decade of data collection and statistical manipulations, a committee of the National Standards Institute made recommendations regarding load factors and load combinations for minimum design loads on buildings (ANSI 1982). The recommendations included different load factors and more load combinations than ACI318. LRFD regulations adopted the ANSI recommendations.

The basic logic for ACI318 and LRFD is identical. Applications differ.

COMPOSITE COLUMN APPLICATIONS REQUIRE DESIGN RULES

Composite columns have found use efficiently for highrise buildings. Steel shapes adequate to support construction loads of concrete floors on steel floor framing can be erected as rapidly as for any steel frame. Subsequent encasement of the steel columns to serve as composite members provides beneficial column stiffness and helps carry total vertical loads in highrise structures. The system also allows more flexibility in construction schedules as well.

Concrete-filled steel tube composite columns have found application as very large vertical load components of highrise buildings. The steel tube walls serve as forms for concrete and provide the most efficient use of steel for stiffness as well as confinement of concrete. When used in very large size columns, the strength of the steel encasement is less significant than is its stiffness.

The structural steel and reinforced concrete families were brought together to form a Structural Specifications Liaison Committee in 1974, specifically to provide guidance for both families again to recognize the estranged entity, composite columns. It had been demonstrated that the concepts of Allowable Stress Design for structural steel could be modified to include strength and stiffness characteristics of reinforcing bars and concrete in order to produce design guidance for composite columns (Furlong, 1976). Modifications to the ASD parameters were calibrated to produce results similar

to those obtained from ACI318 procedures applied to the same composite columns. Since the calibrations were based on results from strength design procedures, they were readily adapted into the SSLC Committee report (SSLC, 1979) and later into the LRFD rules that appeared in 1986.

In a harmonious and logical family, design rules from the structural steel family and design rules from the reinforced concrete family applied to the same conditions of design for a composite column would produce identical results.

Composite Column Design Applications Highlight Family Differences

Material stiffness for structural steel in the elastic range is a constant amount regardless of yield strength for the steel. Inelastic buckling of axially loaded steel columns has been formulated reliably, as has the determination of flexural capacity to include effects of lateral torsional buckling of beams. LRFD interaction functions for steel capacity are taken as the sum of the axial force ratio and the flexural force ratio between limit load and limit strength. LRFD moment magnification does not incorporate an allowance for stiffness reliability.

Abundant torsional stiffness, characteristic for concrete columns, eliminates any ACI318 concern for lateral torsional buckling. Since concrete stiffness varies with concrete strength, ACI318 slenderness provisions have had to incorporate adjustments in the effective stiffness of cross sections composed of steel bars and concrete, acknowledging also the probable cracking of concrete near any limit state of loading. Also, the ACI318 formulation of slenderness effects had to incorporate further adjustments for sustained load creep. ACI318 interaction functions apply directly for cross section strength to resist axial load plus moments that have been augmented for slenderness effects. ACI318 rules require that columns be designed for moment in addition to axial force regardless of framing conditions at beams. Consequently, ACI318 employs a more elaborate evaluation of cross section strength than that required by the LRFD specification.

Resolution of Major Discrepancies Can Promote Unified Design

The application of ACI318 and LRFD rules to composite column design leads to the following significantly different results:

1. The ACI318 insertion of its column reduction factor as a stiffness reliability factor in the moment magnification function ($P_u / \phi P_c$ rather than P_u / P_E as for steel) tends to exaggerate harmful effects of both member and frame instability. The absence of any reliability factor for stiffness in LRFD moment magnification functions is justified by the constant value of steel stiffness prior to the development of inelastic member response. Composite columns probably experience more non-linear response than plain steel columns as the limit state approaches, such that LRFD rules may be marginally less reliable for composite than for plain steel columns. Composite columns always contain more longitudinal steel than plain reinforced concrete columns, such that creep and sensitivity to variable concrete stiffness are dampened significantly. Also, concrete

stiffness itself is not a linear function of concrete strength as implied by the use of the full column reduction factor in ACI318 moment magnification functions. ACI318 rules applied to composite columns can be cited for serious "stability overkill".

2. The fundamental LRFD concept of columns as slender, axially loaded members is a condition almost impossible to achieve in structures with composite concrete slabs and composite columns. The ACI318 requirement of an effective limit axial force lower than the cross section capacity is a reasonable design requirement for composite columns.

3. The simplistic LRFD equations of thrust-moment strength limits are more convenient than they are precise for composite cross sections. The LRFD equations underestimate cross section strength of composite columns. LRFD rules do permit the use of more sophisticated cross section strength estimates such as plastic analysis, but no guidance is given for applications different from the interaction equations. Analytic sophistication with computer support or design aids applicable under ACI318 rules should be referenced and accepted if not incorporated into LRFD rules.

4. The most significant difference among steel and concrete design load factors is the value of 1.20 dead loads in LRFD and 1.40 dead loads for ACI318. Although designers can estimate the magnitude of dead loads to an accuracy within 20% of the actual magnitude, the dead load factor should contain more than the statistical probability of accuracy. For example, if the deformation of unshored steel supports during casting of concrete slabs or substitutions of alternate floor cover, ceiling treatment or utilities caused estimates of dead load to be underestimated by 10% to 15%, the remaining real load factor in the constructed system would be only 5% to 8%. The dead load factor should contain an additional allowance for such unpredictable inaccuracies. The dead load factor of 1.20 seems too low for composite systems. The 1.40 factor of ACI318 may be higher than necessary, but it does create a more desirable safe load spectrum.

The family of construction materials need have no estranged components. It may be desirable to revive a structural specifications liaison group now that strength design and the rationalization of slenderness effects are familiar both to the family of structural steel and to the family of reinforced concrete. As differences in rules for composite columns are brought together, extensions applicable to composite walls and other vertical load composite systems can grow from a common base.

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TECHNICAL PROMOTION OF STEEL CONSTRUCTION IN THE U.S.A.

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ABSTRACT

Steel producers in the United States are achieving quantum improvements in production efficiencies for structural shapes. Technical promotion of steel construction is based on the use of economical steels through the application of Load and Resistance Factor Design (LRFD). In addition, design guides cover a wide range of subjects from column base plates, to roofs of industrial buildings.

AISC computer software includes a connection expert (CONXPRT) program that utilizes the knowledge base of AISC's leading fabricators for the automated design connections. The ELRFD program provides a computer representation of the LRFD Specification.

In the field of bridge construction, plastic design is leading to more economical use of rolled shapes. The optimum design of plate girders is now within reach of every structural engineer through the availability of the PC version of program SIMON.

INTRODUCTION

Since its founding in 1921, the main objectives of the American Institute of Steel Construction, Inc. (AISC) have included the development of design specifications and manuals of steel construction for structural steel buildings. These activities are based on the application of products provided by the steel producers. In recent years, significant reductions in the cost of structural shapes for buildings have improved the competitiveness of steel. The introduction of Load and Resistance Factor Design (LRFD) made steel design more rational because it involves explicit consideration of limit states, multiple load factors and resistance factors, and implicit probabilistic determination of reliability.

To bridge the gap between research and design, AISC is publishing a series of design guides. These guides synthesize research on a given subject and develop recommendations for application to design. The initial series covers a wide range of subjects, such as column base plates, steel and composite beams with web openings, serviceability considerations for low-rise buildings, extended end-plate moment connections, and economical design of low- and medium-rise buildings.

Computer software is playing an increasingly important part in the design of buildings. Numerous programs are available from commercial sources for frame and finite-element analysis and for the optimum design of the framing systems. AISC has concentrated its development of software in areas that have traditionally been the purview of AISC manuals and other design aids. The connection expert (CONXPRT) program utilizes the knowledge base of AISC's leading fabricators for the automated design of several shear and moment connections. Another important software package being developed will provide a computer representation of the LRFD Specification on the basis of the decision tables originally used to organize the specification.

In the field of bridge construction, a breakthrough was achieved by developing a plastic design method for short-span continuous beams. By defining an effective plastic moment, plastic design can be extended to shapes with elements that are more slender than those of the traditional compact shapes. Recent research indicates that the method can be extended further to plate girders. These advances lead to more economical fabrication of bridge members.

Although competition with other materials is intense, the new developments give steel bright future.

STRUCTURAL STEELS

During the decades of the fifties and sixties, the steel industry in the United States was engaged in an intensive effort to develop higher strength steels for structural applications. This resulted in an almost overwhelming number of proprietary steels. A task group of the Column Research Council (now Structural Stability Research Council) [1] suggested a classification of steels for structures that identified the major types of steels and analyzed trends for future development. The task group recommended a rational progression of steels with yield stresses of 36, 50, 70 and 100 ksi (250, 345, 485 and 690 N/mm^2). In addition, it strongly recommended that the steels in each category maintain the specified minimum yield stresses over a wide range of thicknesses. The traditional approach had been to develop structural steels with a given chemical composition but with yield stresses that decreased with increasing thicknesses. For example, ASTM A441 steel had a yield point of 50 ksi (345 N/mm^2) for thicknesses of 3/4 in. (19mm) and under, 46 ksi (317 N/mm^2) for the range over 3/4 to 1-1/2 in. (19 to 38mm), and 42 ksi (290 N/mm^2) for the range over 1-1/2 in. to 4 in. (38 to 100mm). The steel industry responded favorably to this recommendation. A441 steel has now been replaced by A572 Grade 50 which has a constant yield stress of 50 ksi (345 N/mm^2) for thicknesses to 4 in. (100mm). Table 1 shows the current categories of steels and the ASTM steels representative of these categories.

TABLE 1. CATEGORIES OF STRUCTURAL STEELS

Category	ASTM Designation	Yield Stress ksi (N/mm^2)
Structural Carbon	A36	36 (250)
High-Strength Low-Alloy	A572 Gr. 50 A588	50 (345) 50 (345)
Quenched & Tempered Low-Alloy	A852	70 (485)
Quenched & Tempered Alloy	A514	100 (690)

A more complete discussion of these steels was presented at the Second Pacific Structural Steel Conference [2]. Because the current quenched and tempered alloy steels have a relatively high ratio of yield to tensile strength, the AISC Specification does not permit plastic design for steels with yield points over 65 ksi (450 N/mm^2) However, new types of high performance steels should be able to overcome this limitation in the future.

Over the years, the cost of structural steels has gradually increased with inflation. In 1961, A572 Grade 50 (345) was identified as the most economical steel for most structural members [3]. The typical price of this steel in U. S. currency was 6.5 cents/lbs. (14.3 cents/kg) With inflation, the price recently had increased to about 25 cents/lbs (55 cents/kg) for most high strength low-alloy steel products such as plates and shapes. However, a remarkable development has taken place in the production of structural shapes with weights up to about 160 lbs/ft. (mass to about 240 kg/m). Quantum improvements in production efficiencies of these products have been achieved as a result of a shift from integrated mills to scrap-based continuous casting and rolling facilities. Shapes are now being sold at typical prices of 17.5 cents/lbs. (38.5 cents/kg) for 50 ksi (345 N/mm^2) steel. This gives steel a significant advance in building construction compared with reinforced concrete.

LOAD AND RESISTANCE FACTOR DESIGN

One of the significant advances in structural steel design in the United States during the past decade was the implementation of Load and Resistance Factor Design. LRFD is consistent with a worldwide trend towards limit states design and involves explicit consideration of limit states, multiple load factors and resistance factors, and implicit probabilistic determination of reliability. The early research was presented in the 1981 AISC Higgins Award Paper by Galambos [4]. He traces the basic notion of this quantitative probabilistic approach to structural design in North America to the work of Freudenthal [5]. At the same time, the idea of using

multiple factors was suggested in England by Pugsley [6] and a specification using them was formulated in the former Soviet Union. The work of Galambos resulted in proposed criteria for LRFD of steel building structures [7]. It should be mentioned that in North America a design specification based on the limit states approach had already been published in Canada [8].

In the United States, the specifications for the design of structural steel buildings have evolved through numerous versions from the first edition, which was published by AISC in 1923. AISC has traditionally sponsored the development of these specifications by following consensus procedures in the work of the Committee on Specifications. The Committee consists of approximately equal numbers of engineers engaged in research and education, engineers in private practice, and engineers employed by industry. In addition, representatives of government agencies and building code authorities participate in the work. The Committee used the Galambos Bulletin [7] as the starting point for the development of a practical document that is easy to use.

In developing the organization of the new LRFD Specification, the Committee benefitted from the work of Goel, Fenves and Gaylord [9], who had used decision-table techniques for adapting the previous allowable stress design specifications to computer-aided design. This made it possible to organize the LRFD Specification not only for easy use in computer-aided design, but also offered a fresh start after the long evolution of the successive editions of allowable stress design. The work culminated in the publication by AISC of the "LRFD Specification for Structural Steel Buildings" in 1986 [10]. Initial experience with the new specification has been favorable although the learning curves of practitioners show a wide range of slopes. However, most universities and colleges have switched or are switching to LRFD in their teaching so that future generation of engineers will be thoroughly familiar with the use of multiple load factors and resistance factors.

As mentioned previously, the switch to LRFD is part of a worldwide trend toward limit states design. Task Committee 167 of the International Standards Organization (ISO) has been an excellent forum for the exchange of information between developers of national standards. The current draft international standard [11] can be considered as a guide for specification writers. Although limit states design principles are essentially the same around the world, implementation shows a wide variation in ease of use.

Although LRFD is considered a new development in the USA, it clearly has a solid foundation in basic theory and research dating back more than 30 years. Also, further development has not stopped. The AISC Committee on Specifications has completed a second edition of the LRFD Specification which will be published in late 1993. A metric edition will be published in 1994 to facilitate the transition of U.S. Federal Government Agencies to the use of SI units of measurement [12].

The conversion to the metric system in the USA will gradually move from the government to the public sector. It will facilitate communication with structural engineers around the globe. AISC and the European Convention for Constructional Steelwork (ECCS) organized

an international workshop on connection design attended by authors from the United States, Canada, Mexico, several countries throughout Europe, the former Soviet Union, and Australia. The resulting text provides the reader with both a knowledge of work performed in other countries and insight into how this work compares with current practice in the reader's country.

In all, fifty papers by 77 authors and co-authors were given and can be categorized in these general topics: bolts; welds and local strength considerations; predesigned and special connections; composite connections; semi-rigid connections; available connections software; global behavior of semi-rigid connections; examples of frame design; and economy of designs. The proceedings of the workshop provide a valuable resource on connection design [13].

DESIGN GUIDES

The AISC specifications are intended to provide a uniform practice in the design of steel-framed buildings. The objective is to provide design criteria for routine use and not to cover infrequently encountered problems which occur in the full range of structural design. The users of the specification are therefore cautioned that professional judgement must be exercised when data or recommendations are applied. It is emphasized that the design of structures is within the scope of expertise of licensed professionals.

To facilitate the design of steel building structures, AISC provides design aids in Manuals of Steel Construction. Through a progression starting with the 1st edition in 1923, the 9th edition of the Allowable Stress Design Manual is now in use. With the introduction of the LRFD Specification, the 1st edition of the LRFD Manual was published in 1986. The 2nd edition is planned for December 1993. In addition to these basic manuals, there is a need for a more rapid implementation of research recommendations. This has led to the preparation of design guides that synthesize the results of research and provide guidance for application to design.

The first guide in this series covers column base plates and was prepared by DeWolf and Ricker [14]. It includes not only design recommendations but also practical aspects of column base selections. The second guide was authored by Darwin [15] on steel and composite beams with web openings. This guide presents a unified approach to the design of structural steel members with web openings. The approach is based on strength criteria rather than allowable stresses, because at working loads, locally high stresses around web openings have little effect on the member's deflection or strength.

Design Guide No. 3 by Fisher and West [16] covers serviceability requirements for low-rise buildings. It was sponsored jointly by the Metal Building Manufacturers' Association (MBMA) and AISC and is devoted almost exclusively to deflections, vibrations and drift as serviceability criteria. Murray [17] authored the fourth guide which presents a literature overview and complete design procedures for extended end-plate moment connections suitable for continuous frame construction. Allison [18] authored the fifth guide which covers the design of low-and medium-rise steel buildings. It provides basic design rules for economy, live load and

bay size selection. For composite floors it demonstrates economy with LRFD and the effects of shored versus unshored construction. It also considers open-web steel joist systems. For wind load design it discusses drift limits and a variety of bracing systems.

In the sixth design guide, Griffis [19] treats the design of W-shapes encased in concrete. The design procedures are based on the LRFD Specification, which requires that the cross sectional area of the steel shape comprise at least four percent of the total composite cross section. This type of construction makes it possible to use a significant amount of steel framing in structures that traditionally might have been built exclusively in reinforced concrete. The rapid erection of light frame will facilitate placement of the concrete. The frame must, of course, be designed to support all construction loads as well as the wet concrete. Because of the low cost of today's steel shapes, composite structural frames should be optimized for maximum economy of material usage.

The most recent design guide by Fisher [20] gives a complete treatise of industrial buildings from roofs to base plates. The purpose of the guide is to provide the industrial building designer with guidelines and design criteria of the design of buildings without cranes, or for buildings with light- to medium-duty cranes.

SOFTWARE

A wide variety of commercial software for the analysis and design of steel building structures is currently available for personal computers. AISC's magazine Modern Steel Construction regularly lists programs that are applicable to fabricated steel. AISC is concentrating its efforts in areas that are specifically related to the design of connections and structural steel components. The prime product is the connection expert (CONXPRT) program developed jointly with Research Engineers, Inc. (Prof. T. M. Murray). The program is based on the knowledge of AISC's most prominent fabricator members. The expert knowledge contained in the program enables the user to design connections as if he were a leading fabricator in the USA. Two modules are currently available. Module I covers the following simple shear connections: framing angles, shear end-plate, and single-plate. Module II covers several moment connections: all welded, flange welded or flange-plate welded, and flange plate bolted. It also designs column web stiffeners and doubler plates when required.

Other AISC software offerings include a data base of section properties, WEBOPEN, which automates the design requirements of Design Guide No 2, and STEMFIRE, which extends fire- resistant designs published by Underwriters Laboratories to additional structural shapes and common protection materials.

A novel computer program, ELRFD, is a joint development with Vidual Edge, Ltd. ELRFD is an electronic representation of the LRFD Specification on the basis of the decision-table technique developed by Goel, Fenves and Gaylord [9]. The decision tables are included in the program as a data base. The shell of the program then makes it possible to browse the

specification or to check compliance of member designs with the specification. Because it is a formal representation of the specification provisions, it eliminates ambiguities that are sometimes found in the written text.

PLASTIC DESIGN OF CONTINUOUS BRIDGES

The design of highway bridges in the United States is covered by specifications published by the American Association of State Highway and Transportation Officials (AASHTO). Traditional design has been based on elastic analysis and design methods which require that the cross sections of bridge members vary along their length so that the capacities approximate the moment diagrams. In an effort to reduce the fabrication cost of steel beams and girders, an attempt was made to apply plastic design procedures to bridge design. However, traditional plastic design requires that the sections be compact. The ISO draft standard [11] as well as many national standards classify compact sections as Class 1. Such sections have the ability to reach the plastic moment and then rotate plastically through an angle required for mechanism formation while continuing to resist the plastic moment. Unfortunately, such compact sections tend to be uneconomical for bridge applications. To overcome this limitation, a procedure was developed that would permit Class 2 sections to be used for plastic design. Class 2 sections can reach the plastic moment but have limited rotation capacity.

The following thought process was used so that Class 2 sections can be calculated as if they were Class 1 sections by replacing the yield strength, f_y by a reduced effective yield point, f_{ye}. ISO requires that for the flange of a Class 1 compact I section the ratio of the outstanding width b to the thickness t shall satisfy

[1] $$b/t \leq 0.32(E/f_y)^{1/2}$$

where E = Modulus of elasticity

If b/t exceeds the limiting value, the section would not be compact for the yield stress f_y. However, if the member were made of a lower strength material the section could be compact. The yield stress for which the section becomes compact is defined as the effective yield point f_{ye}. The value of f_{ye} can be calculated directly from Equation 1.

[2] $$f_{ye} = 0.102\ E\ (t/b)^2 \leq f_y$$

Similarly, the web depth to thickness ratio, d/t, of a Class 1 section must satisfy

[3] $$d/t \leq 2.5(E/f_y)^{1/2}$$

If the critical d/t ratio is exceeded, the effective yield stress of the web is

[4] $$f_{ye} = 6.25\ E\ (t/d)^2$$

Section A.8.3.2 of the ISO draft standard states that Class 2 I-sections in continuous beams may be calculated as Class 1 sections provided the yield stress f_y is replaced by the effective yield stresses given by Equations (2) and (4). The beams can then be treated as hybrid beams for the purpose of calculating the effective plastic moment.

The above simple thought process was originally proposed by Carskaddan and published by Haaijer, Carskaddan and Grubb [21]. The strength of continuous bridge members according to the effective plastic moment concept significantly exceeded the load corresponding to first yielding of the most highly stressed member. In fact, yielding can occur at service loads. Procedures were therefore developed to permit local yielding at service load and to account for the effect of automatic redistribution of moments at service loads. The complete design method was therefore called Autostress design. The background was summarized in AISI Bulletin No. 29 [22].

AASHTO adopted the procedures in a Guide Specification in 1986, which was recently updated [23]. Several states have built so called Autostress bridges, including New York, Tennessee, Illinois and Maine. Grubb reported significant cost savings, especially for welded beams that could be tailored for composite construction [24]. At the 1992 Transportation Research Board meeting, Wasserman reported in detail on the Tennessee experience [25]. He stated that Autostress goes a long way toward fitting the need for a non-labor intensive quickly fabricated bridge, which is virtually free of fatigue generating details.

PLATE GIRDER BRIDGES

For bridge spans that exceed the capabilities of rolled shapes, plate girders are most economically designed by the Load Factor Design (LFD) method of AASHTO. Detailed design procedures are presented in the AISC "Highway Structures Design Handbook." However, the selection of the optimum girder depth and spacing often requires many iterations. For many years this tedious work was made easy through the use of program SIMON which could be accessed on mainframe computers of U.S. Steel. Through arrangements with the Wisconsin Department of Transportation, the program was also made available to all interested states. Program SIMON has now been converted to run on IBM or compatible personal computers. AISC will soon start marketing this program to supplement the "Highway Structures Design Handbook", bringing this powerful computer tool within reach of every structural engineer.

CONCLUSION

The advances that are being made in manufacturing and design technology demonstrate that steel is alive and well and shows promise for additional improvements in the future. The new developments greatly improve steel's competitive position.

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EXTERIOR WALL ASSEMBLIES FOR EXTREME CLIMATES

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ABSTRACT

Exterior wall assemblies in severe environments pose special problems and require unique solutions. The assemblies are affected by the speed of construction, the choice of structural systems, the use and occupancy and other factors such as tolerances and moisture movements. The choice of system and the arrangement of its components in many cases influence the durability and serviceability of the cladding. This paper provides an overview of factors which affect the performance of cladding systems in severe climatic conditions and provides some guidelines as to the importance of these factors to the structural engineer designing such assemblies.

INTRODUCTION

The main function of the exterior wall and its materials, components and assemblies, is to provide a level of environmental separation between environmentally dissimilar spaces; to provide acceptable resistance to environmental loads such as air leakage, vapour diffusion, the ingress of precipitation or moisture from the ground, and heat transfer. Concurrently, the wall must also perform as a safe and serviceable structure resisting or accommodating the impact of structural loads including wind and seismic loadings, accidental impact, thermal and moisture movements, and elastic and inelastic frame deflections and deformations.

For the purpose of this paper, extreme climates are defined as those climates where the ambient temperature is very high or very low for a prolonged period of time throughout the year, or where temperature variations between the seasons are high. The range in temperature for a geographic location is a function not only of latitude, but also of proximity to major bodies of water, and of prevailing winds. Most areas of Canada and the northern part of the United States are considered to have extreme climates. Extreme climates ranges impose unique problems where wall assemblies are required to provide a high level of environmental separation. In addition, the effects of extreme climates on wall

components and assemblies may be exacerbated by high exposure to precipitation combined with driving rain, repeated freeze-thaw cycling, and corrosive environments resulting from industrial pollution. Corrosive environments are placing special demands on cladding systems. Careful consideration by the designer of the properties of the building materials and components is needed to ensure that wall components and assemblies meet their expected design service life under the service environment.

The choice of exterior wall system should be made only after careful consideration of many factors, including: budget; occupant use; construction schedule; macro- and micro-service environment; structural serviceability, fire, and health and safety considerations; predicted service life of the component and assemblies; the design service life of the building and its components and assemblies; and aesthetics. In general, construction costs are reduced for a system where an individual wall component can be relied upon to provide satisfactory resistance to multiple environmental loadings and structural loads, rather than provide resistance to a single environmental or structural load, and where the designer considers the ease of constructabililty of the wall. Repair and maintenance costs can be reduced through the selection of material and component types having a long service life, by effort to control the micro-environment to which the material or component will be subjected, and by appropriate location of components for ease of access for repair. Operation costs are reduced where the wall provides good resistance to heat transfer, air leakage, vapour diffusion, and moisture ingress.

In this paper, the factors which affect the performance of exterior walls with respect to durability, serviceability and buildability are examined, and recommendations are made as to the desirable features of these walls from the perspective of the structural engineer.

COMPONENTS OF THE WALL ASSEMBLY

To effectively resist or accommodate the imposed environmental and structural loads, a typical exterior wall, above grade, consists of the following components: an exposed, exterior weathering surface necessary to resist the effects of deterioration from precipitation and chemical/biological attack, and to protect the inner components from the affects of agents of deterioration; behind the weathering surface, and in direct contact with the air barrier system, sufficient thermal insulating materials to provide the appropriate level of conditioned interior space, to prevent or minimise condensation on and within components and assemblies located to the interior, and to minimise thermal movements of components within the wall assembly and of the structural frame; a continuous, air-tight, air barrier system resistant to the effects of wind loading (supported by the structural backing), to minimise movement of air through the building envelope which can affect HVAC and indoor air quality, and the amount of moisture deposition on wall components and assemblies; a structural backing to support all the components of the wall assembly; and a vapour barrier to resist the movement of vapour by diffusion through the wall assembly, positioned with respect to the insulation to minimise condensation within the wall assembly.

STRUCTURAL DESIGN REQUIREMENTS

The structural design of the wall assemblies requires the consideration of the many factors, which if not appropriately addressed, can directly result in serious and structural distress to the wall assembly, but most often, will weaken the effective resistance to environmental loads of components and assemblies within the wall, change the anticipated micro-environment of the component or assembly or of adjacent components and assemblies, reduce the service life of the component or assembly, and thereby cause premature deterioration of the wall. At the time of construction, the exterior wall system generally accounts for about 15 to 18% of the cost of the building. Improper design can result in premature deterioration with associated maintenance and repair costs many times the initial costs. Factors affecting the structural design of the wall assembly include:

i) frame shortening and deflections of the supporting elements
ii) lateral loads
iii) thermal movements
iv) tolerances
v) moisture migration

I. Frame Shortening and Deflection of Support Elements

The effects of frame shortening and the deflection of the supporting elements should be accommodated by the connections between the structural frame and the exterior wall assembly. With a concrete frame structure, for example, it is not unusual to expect the frame to be shortened under elastic, shrinkage, and creep effects by 3 to 5 mm per storey, and to expect edge beam and slab deflections (long-term + short-term) in the order of 12 to 16 mm over a span of 5 m.

Figure 1 shows an exterior wall assembly which incorporates a deflection accommodating connection to the supporting concrete slab. The deflection space between the underside of the brick shelf angle and the top of the brick panel wall beneath is provided with sufficient width to accommodate frame shortening, moisture and temperature movements in the brick panel wall, and the movement capabilities of the type of the physical properties of the materials employed in the structural frame and the wall assembly, imposed structural loadings, and the sequence of construction (which affects the magnitude of "residual" deformations and deflections). Connections must not only satisfy structural requirements, but effectively resist, or minimise the impact on other components which are intended to resist, environmental loads.

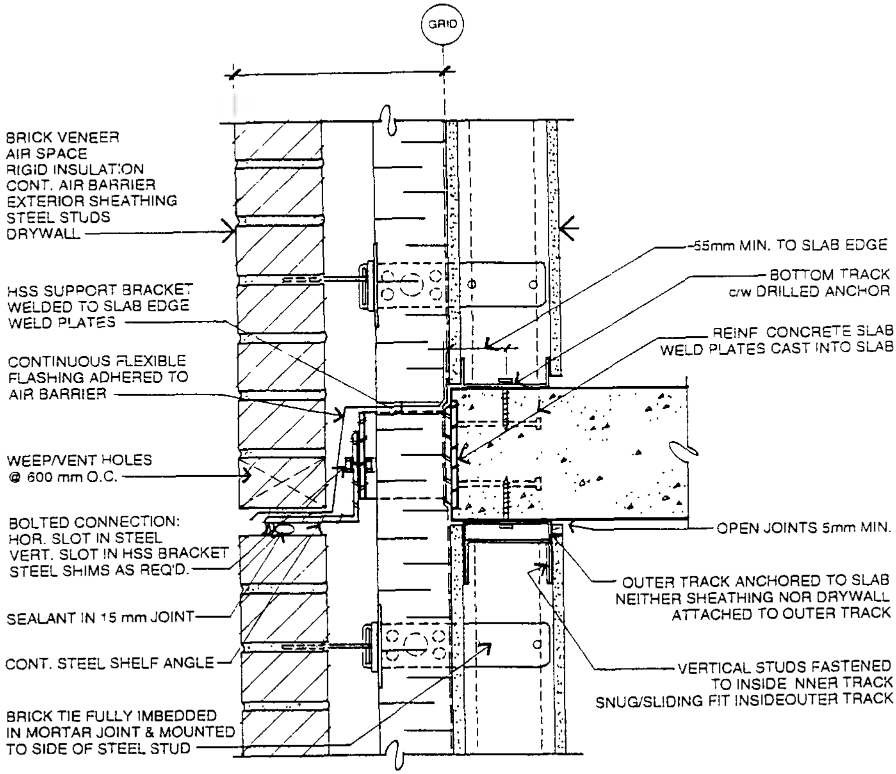

Figure 1. Brick Veneer/Steel Stud Detail At Slab Edge

In severe climates, and because of cost consideration, it is desirable to close in the structure as quickly as possible. The installation of the cladding often begins as soon as floors are structurally able to receive the structural loads from the exterior walls. The speed and schedule of construction thus affects the deformation which must be accommodated by the exterior and interiors walls. Incorporation of horizontal control joints at regular intervals along the height of the building are designed with structural input to safeguard against accidental loading due to deformation and deflection.

II. Lateral Loads

Laterals loads acting on the wall are transferred to the main structural elements by means of connectors, (anchors, ties, and fasteners) the design of which requires due consideration of the limitations of the assembly as it relates to the number of penetrations of the air barrier and the location of fasteners in relation to the structure. Factors such as edge distance and location influence the choice of connectors used to secure the assemblies.

Figure 1 shows a drilled in insert structurally connecting the outer track of a steel stud backup wall to a concrete slab.

III Thermal Movements

Thermal movements become more important in extreme climates because of the extreme temperature fluctuation and the resulting difference between the interior and the exterior environments. Variations as high as 45° to 50° C. can be expected. The effect of large temperature fluctuations is more detrimental when the components are acting in shear and are not separated by an air space and have no provisions in the design to accommodate these movements or to restrain them.

Figure 2 shows a wall assembly partially restrained to resist differential movements between the exterior wythe and the interior wythe. This assembly and the one shown in Figure 1 incorporate air space which allows for the weathering surface to provide resistance to moisture and allow for dry drainage.

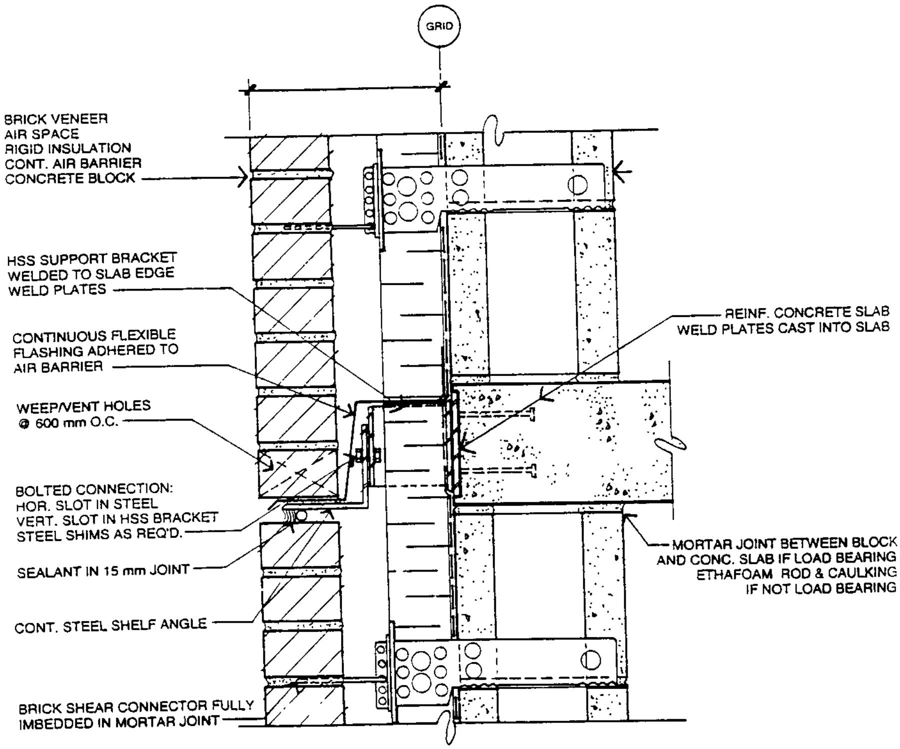

Figure 2. Brick Veneer/Concrete Block Detail At Slab Edge

IV. Tolerances

Tolerances affect the performance of exterior wall assemblies because in order to be accommodated, the construction of the wall as designed is made in many cases impossible.

Tolerance values, specified by code authorities, set the deviation of particular dimensions to be within certain allowable boundaries. Tolerance values are set for individual member dimensions and dimensions of constructed buildings for variations from plumb and level, and for variations from grid and elevation.

It is realistic to expect some deviation from dimensions set by designers on their drawings when a structure is erected. Designers must bear in mind that such deviations could affect the design strength of a system and components of the system. The designer must also be prepared for the situation when the tolerance values specified by the codes are exceeded when a particular member is manufactured and erected. Also, in a system such as an exterior wall assembly, is there correlation between the specified tolerance values for the different components of the assembly? These issues or concerns must be considered by the designers during design and construction of the exterior walls.

The effects of construction tolerances on the exterior wall assemblies are summarised as follows:

a) Increasing or decreasing the cavity with
b) Reducing the insulation thickness
c) Reducing thickness of the veneer and cutting of the shelf angle
d) Increasing the eccentricity on the shelf angle
e) Contouring the wall, resulting in exceeding the specified wall tolerance
f) Changing the behaviour of the tie system
g) Reducing the life expectancy of the wall assembly

V. Moisture Migration

In extreme climates, moisture migration through the exterior wall assemblies requires careful evaluation. In Canada it is safe to assume that in most buildings there is more moisture trying to get out from the interior than entering the closure from the outside. The performance of the assembly can be enhanced by providing for resistance to moisture from entering from the exterior. Incorporation of an air space between the weathering component and the insulation and the installation of flashing at floor levels (as shown in Figures 1 and 2) will ensure that the interior portion of the assembly is pre-treated from the rain.

Evaluation of the due point and ensuring that no condensation will occur in the backup assembly will safeguard against corrosion and deterioration of interior finishes. Since the due point is a function of the difference in temperature between the exterior and interior and the relative humidity, it is impossible to design an exterior wall assembly without having condensation occur at times within the wall. Provided that the material can discharge the moisture back into the space within a reasonable time (at least once a year)

and it is not greatly affected by the pressure of such moisture, the wall is considered adequate. When condensation is expected to cause deterioration then appropriate materials must be chosen.

CONTINUITY REQUIREMENTS

The exterior wall assembly in severe climates must be integrated with the other components and functions of the structure. The connections details to the roof, the parapets and the foundation details require understanding of the concepts of building physics. Figure 3 shows a parapet detail which illustrates the continuity of the air barrier and its connection to the roof. For a high parapet (more than 600 mm) this detail may not be appropriate in buildings with high humidity levels, as moisture may condense at the top of the cold parapet and cause failure. Figure 4 shows an appropriate detail for high parapets.

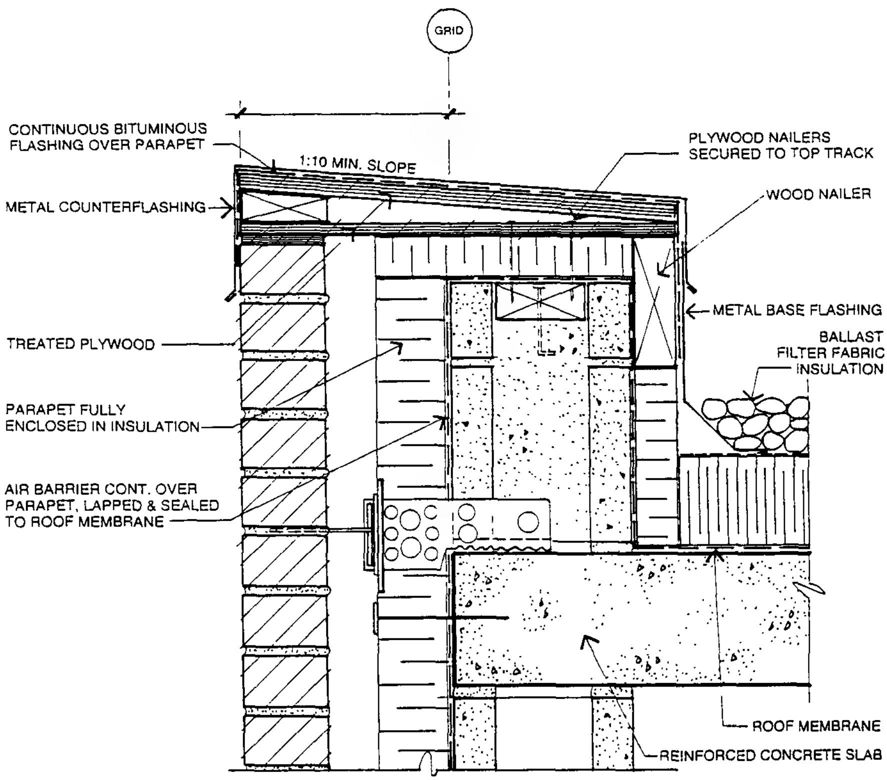

Figure 3. Brick Veneer/Concrete Block Detail At Low Parapet

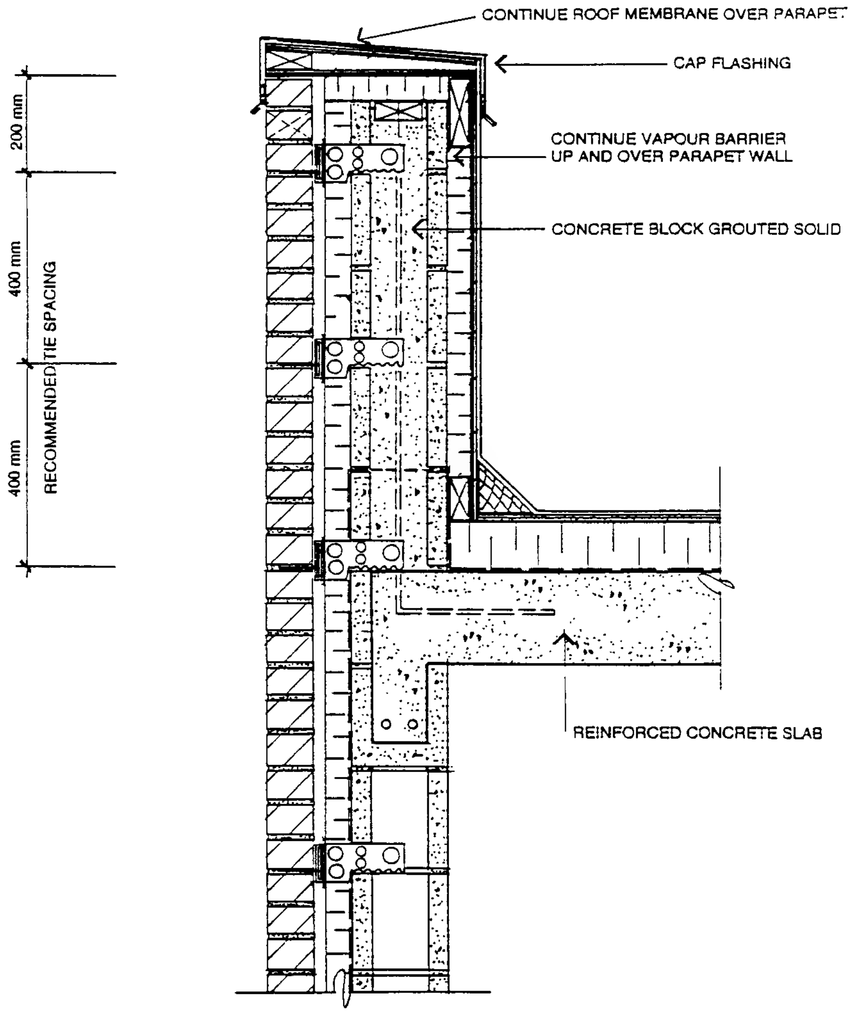

Figure 4. Brick Veneer/Concrete Block Detail At High Parapet

The support of the cladding at intermediate floor level requires details designed to facilitate the continuity of the insulation and the air vapour barrier (shown in Figure 1 and 2). The details at the termination of the exterior wall at the main floor are provided with similar provisions as those at floor levels to ensure continuity of the air barrier and the installation.

Figure 5 shows a detail at the foundation level which incorporates the requirement set above.

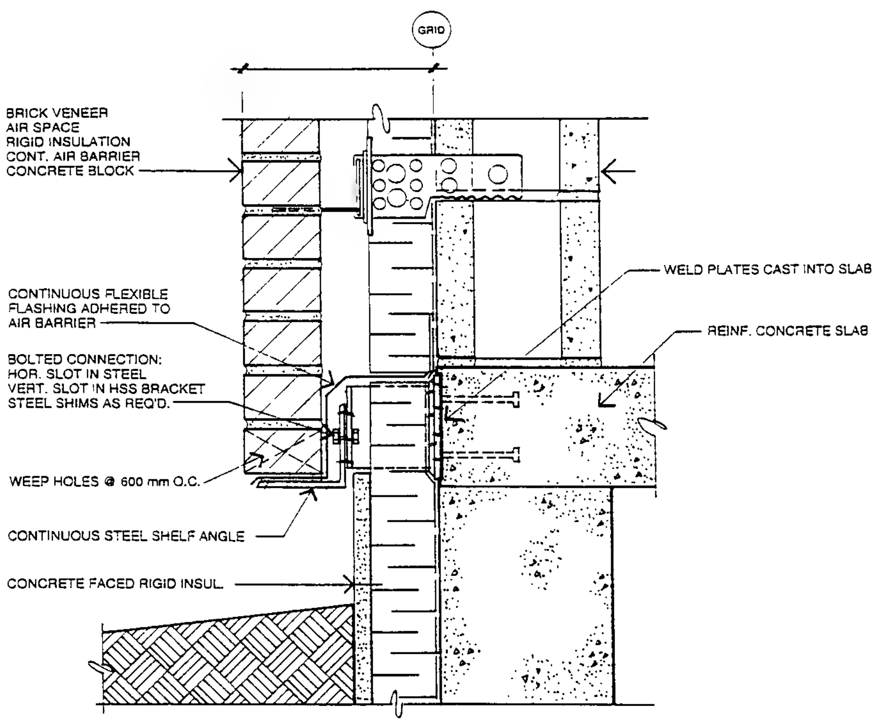

Figure 5. Brick Veneer/Concrete Block Detail At Foundation

Penetrations through the exterior walls is a major source of potential problems. Proper installation of vents requires careful examination of the sequence of construction and due consideration of air leakage through the assembly. Discharge of conditioned air into the cavity can be very detrimental to the system.

Figure 6 shows a typical venting detail of an exterior wall assembly.

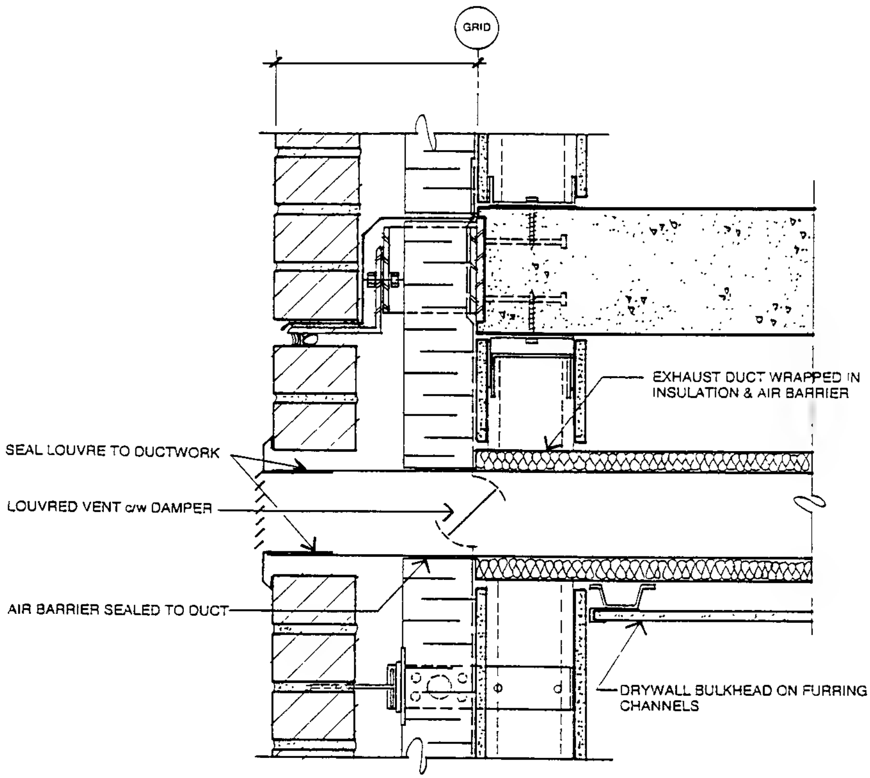

Figure 6. Brick Veneer/Steel Stud Exhaust Vent Detail

CONCLUSIONS

Exterior wall assemblies in severe climates require special design consideration. Failure to address issues such as frame shortening, deflection of supporting elements, effects of loads, thermal movements, tolerances and moisture migration can result in expensive repairs. Research work at the University of Alberta during the last 20 years has made substantial contribution to the understanding and design of exterior walls.

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ENGINEERS, ARTIFACTS AND MONUMENTS

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ABSTRACT

It can be argued that the greatest revolutionaries of our time are the engineers who have ushered in the urban industrial society in which we live. The newly established and related fields of the history of technology and industrial archaeology seek to explain the Industrial Revolution which has been a dominate theme in the history of western civilization for more than two centuries. With these ideas as background, the multidisciplinary aspects of these new fields are described including the role of engineers in the history of technology, industrial archaeology, and the preservation of engineering works.

INTRODUCTION

When one thinks of artifacts and monuments it is usually in terms of museum exhibitions or historic architecture or even perhaps the fruits of an archaeological "dig," but rarely in terms of engineering works. If, however, one views the Industrial Revolution as a defining event in modern history then the great public works, industrial processes, and transportation systems of the eighteenth, nineteenth and early twentieth centuries created by engineers are worthy of study , interpretation and preservation. Such activities constitute, at least, an expanded vision of the Liberal Arts and perhaps the integration of the history of science and technology into the Humanities which would result in the establishment of a New Humanities. In fact, the pioneering work of Sarton (1937) was published under the title History of Science and the New Humanities as early as 1937. Thus the history of science and technology, can provide a meaningful bridge between the two cultures, (science, technology, and the Liberal Arts) proclaimed in C.P. Snow's 1959 Rede Lecture at Cambridge University (Snow, 1969).

In a larger sense in a rapidly changing world society our understanding of the influence of science and technology in the past may well serve as a guide to the future. I would hasten to add, however, that it is not the task of the historian to be a futurist and to extrapolate from past events into the future course of history. A knowledge of history, nevertheless, provides a rich background for decision making which may influence the future.

If one takes this broader view of the role of technology in history and takes seriously the artifact as evidence then, I believe, one is obligated to have an integrated knowledge of both disciplines. It is rather like Rudyard Kipling's royal marine who was a soldier and sailor too. In order to grasp how engineering, history and archaeology can be integrated in an interdisciplinary approach, it is necessary to explore these contributing disciplines.

HISTORY OF TECHNOLOGY

The history of technology is one component of the general field of history. History can be used as a means to achieve certain goals. At the beginning of the nineteenth century it was used by various authors to inculcate feelings of national pride or the use of the past as evidence for a moralistic interpretation of human activities. To overcome this lack of objectivity, historians in the latter half of the nineteenth century turned towards what they considered an objective approach to writing history free of any sentimentality, moralistic diatribes or any attempt to foster national pride. The entire case of this concern for objective history is beautifully stated in Charles Dickens' Hard Times when Mr. Gradgrind states his philosophy of education.

> "Now, what I want is, Facts, Teach these boys and girls nothing but Facts. Facts alone are wanted in life. Plant nothing else, and root out everything else. You can only form the minds of reasoning animals upon Facts: nothing else will ever be of any service to them. This is the principle on which I bring up my own children, and this is the principle on which I bring up these children. Stick to Facts, sir!" (Dickens, 1965)

Applied to history, this approach championed the idea that the principal task of the historian was to organize the information in a chronological order and let the facts speak for themselves or as Harry Truman said, "tell it like it is."

Perhaps the extreme case of Victorian confidence was Lord Acton's report of 1896 to Cambridge University Press on his multi-volume work entitled Cambridge Modern History.

> "It is a unique opportunity of recording, in the way most useful to the greatest number, the fullness of the knowledge which the nineteenth century is about to bequeath. . . . By the judicious division of labour we should be able to do it, and to bring home to every man the last document, and the ripest

> conclusions of international research.
>
> Ultimate history we cannot have in this generation; but we can dispose of conventional history, and show the point we have reached on the road from one to the other, now that all information is within reach, and every problem has become capable of solution. (Acton, 1907)

The implication in Acton's work was that the facts will lead to only one conclusion and that definitive history can be written once and for all. Mercifully this approach proved to be a false premise which if it had been correct would have put most historians out of a job.

For those of us engaged in engineering research, it would be the height of folly to suppose that all one had to do was to organize the raw data without evaluation and the research data would speak for themselves. This is precisely the point made by E. H. Carr (1976) in his popular work What is History? quoting the well known Italian historian, Croce that history "consists essentially in seeing the past through the eyes of the present and in the light of its problems and the main work of the historian is not to record but to evaluate".

In order to evaluate historical data, the historian needs an hypothesis or philosophy of history just as the researcher in science and technology needs to have a working hypothesis in order to make sense of his research data. Developing a philosophy of history became a difficult task because many in the English speaking world thought that such a philosophy was not only implicit but that in some sense it was guided by an idea of Victorian progress which was epitomized by the slogan "day by day we are getting better and better," coupled with the fatherhood of God and the brotherhood of man. With these ideas uppermost in peoples' minds, the very thought of an explicit philosophy of history seemed to verge on blasphemy. Carr (1976) writes succinctly that:

> "British historians refused to be drawn, not because they believed that history had no meaning, but because they believed that its meaning was implicit and self-evident. The liberal nineteenth-century view of history had a close affinity with the economic doctrine of laissex-faire also the product of a serene and self-confident outlook on the world. Let everyone get on with his particular job, and the hidden hand would take care of the universal harmony. The facts of history were themselves a demonstration of the supreme fact of a beneficent and apparently infinite progress towards higher things. This was the age of innocence, and historians walked in the Garden of Eden, without a scrap of philosophy to cover them, naked and unashamed before the god of history. Since then, we have known Sin and experienced a Fall; and those

> historians who today pretend to dispense with a philosophy of history are merely trying, vainly and self-consciously, like members of a nudist colony, to recreate the Garden of Eden in their garden suburb. Today the awkward question can no longer be evaded."

Carr is exactly right with regard to the struggle for a philosophy of history. The marxist dialectic has lost favor and in recent times there has been a dearth of historians who have a philosophy based on capitalist economic theory or a christian perspective which endorses the idea of progress. Nevertheless, there are useful hypotheses such as, Thomas Kuhn's idea of paradigm shifts in the history of science or the idea of the frontier in North America history.

The history of technology had its origins in a biographical tradition, clothed in the idea of progress and celebrating history on a heroic basis. In his well known television series Civilisation Sir Kenneth Clark (1969) refers to the engineers responsible for the new urban industrial age in the nineteenth century as the great field marshals of the Industrial Revolution. It was Sir Samuel Smiles (1862 & 1884) who popularized the biographical tradition with his well known books Lives of Engineers and Industrial Biography. It was history on a heroic basis which sought to validate Smiles' philosophy of self-help, which is epitomized by a verse on a Victorian jug.

> Self help is noble schooling
> Always do your best
> and leave the rest
> to God's almighty ruling.

The problem with history of technology, on a heroic basis, is its lack of social, economic, and political context. We may well celebrate the field marshals of the Industrial Revolution, but their work is not very meaningful except in a larger historical context. Wars are not won by field marshals alone and neither were the leading engineers successful apart from legions of workmen, fellow engineers, and industry supplying needed products. Architecture has often been called the most social of the fine arts. In a like manner, great engineering works require large amounts of capital and are usually labor intensive. Thus civil engineering, in particular, is beholden to the society which it serves and as a result reflects its values. Thus to be valid, the history of technology needs to be placed in a broader context.

Compared to more general history, the history of technology is a comparative new comer. Sarton's (1937) monumental work is a landmark in the field and can be credited with establishing the history of science as a discipline. At a later date but in a similar way, a multi-volume work sponsored by Imperial Chemical Industries entitled The History of Technology was published in 1956 under the editorship of Singer (1956) and represents an important milestone in the development of the field. Since then, we have seen books, monographs, and many articles written on the subject. We lack, however, an integrated approach to the field and even more important, publications which could be used as text books for teaching courses on the subject or material which could be integrated into survey courses in nineteenth century British, Canadian and American history

are lacking. As I survey recent historical publications, I find a number of books on feminist and black history and a dearth of material published on the history of technology.

The pioneers in the field were drawn largely from science and engineering and were concerned with what we call internalist matters, that is how science and engineering actually work. They evinced a deep concern for origins of invention and the transmission of technology to individuals and nations, so called technology transfer. There was also an emphasis on leaders in the field of science and engineering who made major contributions to science and/or engineering. In the case of engineering, the History of Technology was concerned not so much with a history of ideas but a history of invention, particularly those technologies and industries which arose during the Industrial Revolution. The pendulum has swung strongly in the other direction with corporate histories, environmental concerns, the influence of technology on society, and questions of context constituting many of the recent themes in the history of technology. One welcomes this broadening of the field, but there is a concern that some of these historians are not royal marines in the sense that they write sometimes about technology which they don't really understand. It is very clear to me that engineers need to be involved in a multidisciplinary approach to the history of technology.

THE ENGINEER AND INDUSTRIAL ARCHAEOLOGY

Following World War II with the loss of empire and the need to play a new role in Europe, many in Britain sought to celebrate the nation's leading role in the industrial revolution by investigating and, in select cases, preserving the physical remains of Britain's early industry and transportation systems. In the best British tradition, industrial archaeology sprang from groups of amateurs interested in specific local industries and particularly in canals and railways. The work of these local groups include published histories and increasingly the preservation and restoration of industrial monuments. These activities include the reopening of abandoned railway lines, restoring rolling stock, especially steam locomotives and the restoration to operating condition of a variety of stationary steam engines used in pumping, milling and other power sources used in textile mills and other industries. Perhaps, the most impressive work has been in the canal system, which by the end of this century will be reopened in its entirety for recreational use. In the British case then this was an amateur, local development, trying to preserve and understand the great industrial revolution in Britain on a local or thematic basis. It found difficulty in establishing a national organization in industrial archaeology, let alone a vigorous discipline in academia. There are certain notable exceptions to this, such as the Iron Bridge Gorge Museum and its relationship with the University of Birmingham.

The Society for Industrial Archaeology in the United States was founded by a group of interested professionals from various fields on a rainy Saturday morning in 1971 at the Smithsonian Institution in Washington. Very few academics were represented, but a spectrum of professionals that were working

in museums and other government agencies. This group then was not a grass roots movement and still is more national in its outlook and activities than its British cousin. It has failed though in the same way to become a vigorous part of academia. One has only to think of the great amateur archaeologists of the 19th century, Heinrich Schliemann, Arthur Evans and others and how these amateurs managed to introduce their interest in the past into academia. I do not think anyone debates that archaeology is a central part of the great Liberal Arts tradition. In fact, what we know of ancient history is largely based on archaeological evidence. Thus, I would say that the history of technology and its closely related field of industrial archaeology have come into their own, not in academia but in the public history area.

During the depths of the Great Depression, Harold Ickes, one of the stalwarts of the New Deal established the Historic American Building Survey of the National Park Service. It had a twofold mission. One was to document the nation's architectural heritage with particular emphasis on high style architecture and the edifices associated with prominent people. Like the New Deal public works program, it was intended to create jobs for unemployed professionals, in this case architects and landscape architects. Following the Second World War a tripartite agreement was signed by the National Park Service, the American Society of Civil Engineers, and the Library of Congress which established the Historic American Engineering Record. Its task was to document historic engineering structures by using the techniques already pioneered by the HABS with strong emphasis on illustrative material in terms of measured drawings and archival photographs. In the HAER program alone, there are more than 30,000 structures and sites in the Library of Congress collection. In my view, the techniques and standards established for documenting structures is one of the United States' great contributions to the historic preservation movement. The documenting of structures by either HABS or HAER has gained increased momentum recently with the implementation of the 1966 National Historic Preservation Act. According to this law, any adverse effect on historic sites threatened with development and using federal funds, must be mitigated in an appropriate way. This often results in a thorough documentation of the site before any construction begins, whether it be simply modifications to or total destruction of the site. Many of these mitigation studies involve historic engineering works. In the United States the historic significance of a site is determined by the listing of that site on the National Register of Historic Places.

The Canadian approach to historic sites is quite different. It is more historical than archaeological in the sense that it takes a broader historical approach to the history of Canada and from these investigations certain sites are identified in the context of national themes, such as transportation, industrialization, agriculture, and urbanization. Thus, a context is first developed and then sites are identified and documented where appropriate. One of the difficulties in the United States at the moment is that by placing the artifact first one is then saddled with determining its significance in a broader context. As an example when I served as president of the Society for Industrial Archaeology, I made it a point to visit each of the chapters in a kind of Elizabethan progression. At most of these chapters, I was greeted with a welcome which

included the phrase "the birth place of the American Industrial Revolution." Because of a lack of context in any of the great industrial themes, such as iron and steel, coal, or transportation, it is difficult, even though sites have been documented by HABS/HAER, to determine their national, state, or local historical significance. One of the most recent trends is to undertake contextual histories of selected industries such as iron and steel or coal. Such work is useful in its own right but is necessary to establish the historical significance of a given site more accurately. At the provincial and municipal level in Canada the approach is much more like that in the United States in that artifacts and industrial monuments are often documented, interpreted, and preserved without the necessity of validating their national significance. They are judged on their intrinsic merit, local interest or simply regional significance.

INDUSTRIAL ARCHAEOLOGY TECHNIQUES

One of the hallmarks of industrial archaeology is its multidisciplinary approach to historic sites. It is very much a team effort often involving historians, engineers, architects, landscape architects and other professionals who are concerned with a sense of place. Not all of these professionals are used on every site nor are all the techniques available to industrial archaeologists employed on every occasion. Nevertheless, the best means of understanding this new discipline is to understand its various components. Although industrial archaeologists have been working for more than a quarter of a century, there is still no recognized single text on industrial archaeology techniques. The exception are several books (Pannel, 1966; Raistrick, 1972) dealing with British industrial archaeology which are quite rudimentary and now very much out of date.

Inventories and Evaluation

Inventories are usually a first step in any archaeological investigation, since without a knowledge of particular sites, it is difficult to know what to document and to what level of detail. In the United States the foremost example of inventories would be the historic bridge inventories and evaluations completed in each of the fifty states under the aegis of the Federal Highway Administration. Unfortunately, there were few regulations to guide this work which was a pioneering effort in the field. Thus, each of the states developed its own inventory and database as well as a method for evaluating the historic significance of bridges within its boundaries. It has been a very large undertaking in West Virginia alone, where there were more than 4,300 bridges which were potentially eligible for the National Register of Historic Places by virtue of being fifty years old or older. The evaluation system which involved the integrity of the structure, the date of its construction, its association with historical events of the site or well known designers were all coupled together to provide a quantitative rating system. If all of this information could have been put on a common database and a unified evaluation system established, it would

be of considerable interest to historians, historic preservationists, planners, as well as highway engineers. This same approach can be used for other themes such as coal, iron and steel and a myriad of other industries.

Written and Graphic Records

Unlike their classical brethren, industrial archaeologists have access to written documents. They, however, are often of a different nature than the material generally used by traditional historians. These include engineering designs and drawings, calculations, articles appearing in engineering and industrial journals, trade catalogues, photographs, and other illustrative material. These constitute the basic raw material which the industrial archaeologist has to deal with. Because public funds were used in many engineering works considered of historic significance, there is usually extensive archival material in the public records at local, state, or provincial and federal levels. All of these sources must be carefully evaluated in preparing a specific history of a given site and yet providing some historical context so that the structure can be evaluated.

Documentation

Having completed the preliminary work in terms of inventories and evaluation, site documentation in terms of measured drawings, archival photographs, and written histories can be undertaken. Not only are historians of technology concerned with context but also with industrial archaeologists, as well as archaeologists in general. V. Gordon Childe, (1968) the former director of the Institute of Archaeology at the University of London, puts it succinctly when he says:

> "The public, I suspect, still thinks of monuments as ivyclad ruins and isolated blocks of stone, carved or inscribed. To many, relics are single coins or flint implements, turned up in ploughing or ditching, if not personal mementoes - a button from Prince Charlie's vest, the joint of a martyr's toe, a tooth of Buddha. None of these, least of all the last group, are likely to be significant archaeological data. To have a meaning that an archaeologist can hope to decipher, an object must be found in context".

The first task is to prepare a site map or, in selected cases, to adapt existing maps to the site so that artifacts and monuments can be carefully located which increases their value for historians and preservationists. Many modern techniques can be used in preparing site maps, not only of its present configuration but of earlier conditions as well. The field of remote sensing can be brought to bear to great advantage on site mapping. This work includes traditional aerial photography, the use of infrared images, and the location of the site with satellite technology. This work is greatly facilitated by new developments in geographical information services (G.I.S.).

One of the central features of site documentation is the preparation of measured drawings. These drawings provide information for historians, preservationists, and those involved in interpreting a particular site. Because funds and time are usually limited on any site documentation, it is important to establish what is being documented and to what level. In addition, it is important that the photographic evidence be carefully integrated with the measured drawings so that a complete picture of the site can be forthcoming.

The traditional approach was to use hand measurements in the field, transcribe these first into pencil drawings and then produce a final archival stable product by committing the information to ink on Mylar. Many of the drawings in the HABS collections are of general illustrative quality, whereas much of the information that is needed in the industrial archaeology field requires a considerable amount of detail. Thus, there is a continuing tension between the illustrative and aesthetic qualities of the drawing on the one hand and the ability of such drawings to convey information on the other. Certain compromises can be effected. In the case of historic bridges, it is possible to have a schedule of members with their dimensions, particularly the cross sections and details of the joints, leaving the drawings to give overall geometric dimensions and other information without cluttering the drawing with a myriad of details. It should be pointed out if the purpose of documenting the site is for preservation purposes then detailed drawings are necessary so that they can be used for contract documents. Such drawings require far greater detail than one would expect in conventional measured drawings according to HAER standards.

The hand measured drawing has been the standard for site documentation for more than half a century. With new technologies developing at a rapid pace, it appears that the ink on Mylar drawings produced by hand are becoming obsolescent. Amongst the new techniques, the oldest of which is close range photogrammetry, are newly developed techniques such as metric and digital cameras. Both of these techniques can be coupled directly to computer assisted design systems (CAD) to produce drawings without the necessity of hand measurements in the field or using traditional drawing techniques. These techniques, of course, have not been developed specifically for historic structures but represent a whole new trend in the production of illustrative material for engineering structures.

For documenting sites photographically, large format view cameras are the standard equipment to produce archival photographs. In the view camera the front lens can be moved vertically or horizontally to correct any distortions which might occur in photographing a building, a bridge, or other structure. Special processing of the negatives and prints is necessary to insure that they will be archivally stable for a period of more than two centuries. This meets the Library of Congress criteria for their collections. To date, all of this work has been done on black and white prints because color photography was not considered to be archivally stable.

In many cases, the most important aspect of a historic site is the process rather than associated structures. For example, the historic LaBelle Nailworks in Wheeling, West Virginia produces hand cut nails on machines which were patented before the American Civil War. Thus, cine-photography may be the

most appropriate way of documenting such a manufacturing process. There was concern that using video cameras would not produce archival stable tape, but it is now possible to digitize video cassettes in which form they should last indefinitely. To supplement the basic measured drawings, photographs, and site history, oral interviews of engineers and workers associated with a particular industry or industrial site are often used especially when an industrial process is being documented. The quality of such interviews depends very much on the interviewer and his knowledge of the technology in question as well as his skills as an rapporteur. The final product should consist of field notes, written and graphic material together with bibliographic sources and other materials which would make the documentation complete.

PRESERVATION OF ENGINEERING WORKS

If monuments of our rich technological past are to be sensitively preserved while at the same time suit modern demands for safety of the public then engineers need to be involved. Preservation activities occur at various levels, ranging from documentation to full restoration according to the highest standards.

Industrial archaeology is concerned primarily with documentation which is a form of preservation. Through site recording, we are preserving a record of the past. Site documentation may also be viewed as the first step in the adaptive reuse of a historic structure or the basis for full restoration.

Adaptive reuse of structures is a growing trend in the historic preservation movement. Such work may entail converting a textile mill into an apartment complex or adeptly reusing a warehouse for office space. Adaptive reuse is viewed by many in the preservation movement as environmentally sensitive and economic feasible rather than new construction. One of the most creative adaptive reuse projects is the conversion of the large Quaker Oats silos in Ohio into attractive apartments. The possibilities are endless and limited only by the imagination of the architect or engineer.

Restoration is a special category where the architect and/or engineer is attempting to return the structure, whether it be a bridge or a building, to its original condition or at the time when some historical event occurred at the site. A good example is the Philippi Covered Bridge in Barbour County, West Virginia. It was constructed by West Virginia's pioneer covered bridge builder Lemuel Chenoweth in 1852. It is a magnificent example of a two-lane, two-span, covered bridge and is possibly the most visited historic site in West Virginia. On June 3, 1861, the first land skirmish of the American Civil War occurred at the bridge site. There was little difficulty in determining that the bridge should be returned to its Civil War condition at the time of the Battle of Philippi. Other structures pose a greater challenge in determining the preservation philosophy, whether to return it a particular period or to interpret a large complex as a continuum over a period of time. Nevertheless, a restoration plan needs to be in place before any architectural or engineering design work is undertaken.

Bridges present an unusual challenge to engineers who must consider the

load carrying capacity of the structure as well as the preservation of its historic components. The rule is to repair rather than replace, based on upon the structural analysis of the loads to be carried by each individual member. With this approach, the structural analysis drives the entire restoration design. There are two basic problems connected with the analysis of historic structures. The first is to determine the physical properties of the materials employed, such as cast and wrought iron. It should be pointed out that these materials are often obsolete, not covered by codes of practice, and, in addition, their properties vary quite widely from structure to structure. In many cases, the preservationist does not have the luxury of removing testing coupons from the structure which would determine physical properties using traditional testing techniques. The second problem is how to model the structure to produce a valid result. The two concerns are joined when the question of safety is considered.

The development of the mushroom slab floor provides a good example of the challenge to engineers. By the beginning of the twentieth century, concrete can be said to have come of age. No longer was it necessary for concrete structures to imitate other forms, using traditional materials, such as timber, iron, or masonry. Structural concrete could be formed in any shape and reinforced to take any tensile stresses which would develop under load. The development of the flat slab floor was not a cooperative effort, but rather competitive and, as a result, spawned a large number of proprietary systems which were covered by patents. These patented systems relied for their claim of uniqueness on special reinforcing bars and/or reinforcing configurations. Since flat slab floor buildings are ideal candidates for adaptive reuse because of their inherent strength, detailed investigations of these proprietary systems could make a significant contribution to the preservation of engineering works. On one project, I applied a yield line analysis to determine the capacity of a flat slab floor in a warehouse which was to be converted into an electrical substation. Fortunately, there were plans available to determine the position and sizes of the reinforcement used. Core borings in selected areas provided the information needed on concrete strength. In the absence of plans for a given floor may be necessary to use nondestructive testing methods to determine the size and location of the reinforcement. Nondestructive testing procedures also offer the possibility of determining the properties of the structural components, regardless of the material used. Knowing the properties of these components other nondestructive testing methods offer the possibility of determining the actual loads in individual members. In the latter case, the results can be compared to a theoretical solution.

Additional examples of historic structural systems might include jack arch brick floors supported on iron beams, cast and wrought iron framing for bridges and buildings and the analysis of timber covered bridges. In each of these cases, there are no standards of practice to guide the engineer in assessing load factors for materials and structural systems since the information needed is not represented in standards of practice or building codes. For both adaptive reuse and restoration of historic structures, engineers need to take seriously the challenge of producing standards of practice based upon both research and experience.

CONCLUSION

One of the hallmarks of a mature profession is a lively concern for its past. Not a chauvinistic vision of past triumphs but rather a sense of continuity and an understanding of the present as a result of past events. If the history of technology and industrial archaeology are to have a secure place as part of our understanding of Western Civilization, it will be imperative that engineers participate more fully in documenting, preserving, and interpreting our rich industrial heritage. It is essential that elements of this heritage be incorporated into mainstream history courses at both the collegiate and pre-collegiate level. Equally important, engineering curricula would be greatly enriched if courses on the history of engineering were made a part of all degree programs. It would provide a bridge between engineering and the Liberal Arts and help to establish a broader vision of the Humanities.

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THE USE OF RITZ VECTORS FOR THE STATIC AND DYNAMIC NONLINEAR ANALYSIS OF STRUCTURAL SYSTEMS

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ABSTRACT

A general numerical method is presented, which is very efficient, for the static and dynamic analysis of linear or nonlinear structural systems with a limited number of predefined nonlinear elements. Stiffness and mass orthogonal Load Dependent Ritz Vectors of the elastic structural system without the nonlinear elements are used to reduce the size of the nonlinear system to be solved. The forces in the nonlinear elements are calculated by iteration at the end of each time or load step. The uncoupled modal equations are solved exactly for each increment with the nonlinear modal forces placed on the right-hand-side of the equations.

Several examples are presented which illustrate the efficiency and accuracy of the method. The computational speed of the new method is compared with the traditional brute-force method of nonlinear analysis. It is shown that for certain types of nonlinear systems the new method can be over 1000 times faster.

The detailed algorithms used for the generation of the load dependent vectors and the exact integration of the modal equations are given. Therefore, the method can be easily incorporated into existing computer programs for linear dynamic analysis.

INTRODUCTION

A large number of very practical structures, when subjected to static or dynamic loading, have a limited number of points or members in which nonlinear behavior takes place. Local buckling of diagonals, uplifting at the foundation, contact between different parts of the structures and yielding of a few elements are examples of such systems. For dynamic loads it is common practice to add concentrated damping, base isolation and other energy dissipation elements to the structure. Figure 1 illustrates some typical types of nonlinear problems which

can be solved by the method presented in this paper. In many cases, these nonlinear elements can be easily identified; for other structures, an initial elastic analysis is required to identify the nonlinear areas. The numerical method for the integration of the modal equations is exact for a linear variation of loads during a time step. Therefore, the method can be used to solve static load problems by using large time increments and large modal damping.

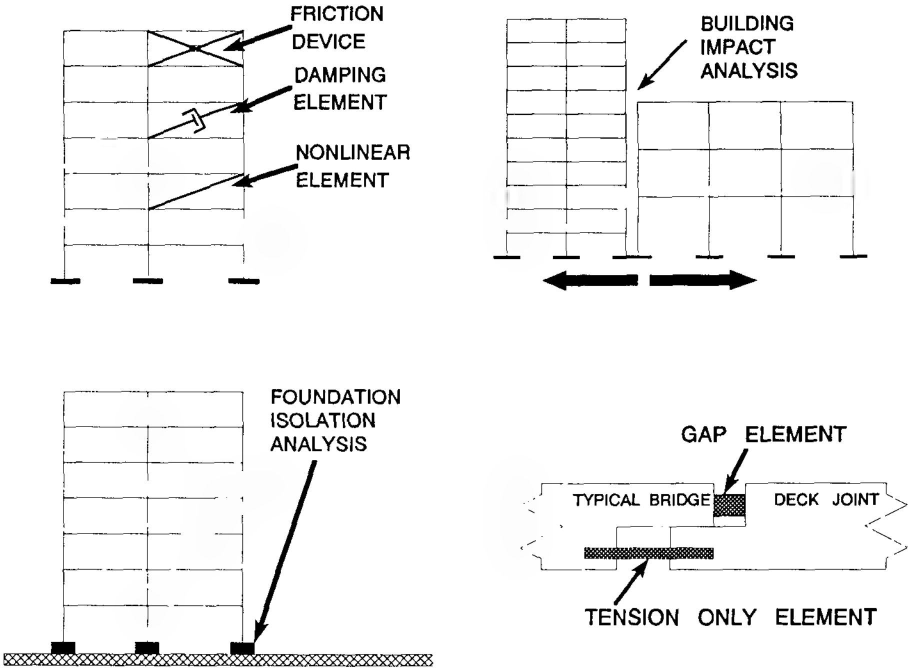


FIG. 1. Typical Nonlinear Elements

The standard approach to the nonlinear dynamic analysis of structures is to use a step by step time integration method in which the stiffness matrix of the complete structure is formed for each time step. In this approach the stiffness matrix requires triangularization for each time increment and iteration is required within each time step. Since the complete system is considered to be nonlinear a large number of equations need be formed and solved for each time step. It is apparent that this method of nonlinear analysis is computationally intensive and can require a significant amount of computer time compared to a linear dynamic analysis of the same structure.

The method presented in this paper is very similar to the approach presented previously (Ibrahimbegovic and Wilson, 1989) in which structural systems with concentrated damping elements, at arbitrary locations, are solved for dynamic loading. Therefore, it is possible to solve linear structures with nonproportional damping, using undamped modeshapes, without needing to use complex frequencies and complex mode shapes. However, since that publication it has been the experience of this author that the use of Load Dependent Ritz Vectors yeilds more accurate results with a significant reduction in computational effort.

For the method presented in this paper, the first step in the analysis is to calculate the Load Dependent Ritz Vectors (Wilson, Yuan and Dickens, 1982) for the computer model without the nonlinear elements. A standard time-history analysis is then performed by the mode superposition method. If nonlinear or damping elements are present the forces in the elements at the end of each time step are calculated. The element loads are converted to modal loads and applied to the right hand side of the modal equations. The modal equations are integrated by a method which is exact for a linear variation of the load during the defined time increment. Iteration is then performed within the time step until the loads at the end of the time increment converge. Since the properties of all nonlinear elements and the modal response at the beginning and at the end of each time step can be stored in high speed computer storage, convergence is very fast.

The method can be easily incorporated into existing linear elastic structural analysis programs. Only the modal integration subprogram need be modified extensively; the pre and post processor for the linear program need not be modified. As in normal linear mode superposition analysis, it is the responsibility of the user to check, with multiple analyses, that a sufficiently small time step and the appropriate number of modes are used. Therefore, the method will converge to the exact solution.

GENERATION OF LOAD DEPENDENT RITZ VECTORS

Several different algorithms for the generation of Load Dependent Ritz Vectors have been previously published since 1982. Therefore, it is necessary to present in the paper the latest version of the method for multiple load conditions as summarized in Table 1.

It is possible to use the exact free-vibration mode shapes to reduce the size of both linear and nonlinear problems. However, this is not the best approach for the following reasons:

1. For large structural systems the solution of the eigenvalue problem for the free-vibration mode shapes and frequencies can require a significant amount of computational effort.

Table 1. Algorithm for the Generation of Load Dependent Ritz Vectors

I. INITIAL CALCULATIONS

A. Triangularize stiffness matrix

$$\boldsymbol{K} = \boldsymbol{L}^T \boldsymbol{D} \boldsymbol{L}$$

B. Solve for block of "b" static displacement $\boldsymbol{u}_s$ vectors due to spatial load patterns $\boldsymbol{f}$

$$\boldsymbol{K u}_s = \boldsymbol{f}$$

C. Make block of vectors $\boldsymbol{u}_s$ stiffness and mass orthogonal, $\boldsymbol{V}_1$

II. GENERATE BLOCKS OF RITZ VECTORS i = 2,....N

A. Solve for block of vectors $\boldsymbol{X}_i$

$$\boldsymbol{K X}_i = \boldsymbol{M V}_{i-1}$$

B. Make block of vectors $\boldsymbol{X}_i$ stiffness and mass orthogonal $\bar{\boldsymbol{V}}_i$

C. Use Modified Gram-Schmidt method (two times) to make $\bar{\boldsymbol{V}}_i$ orthogonal to all previously calculated vectors and normalized so that

$$\boldsymbol{V}_i^T \boldsymbol{M V}_i = \boldsymbol{I}$$

III. MAKE VECTORS STIFFNESS ORTHOGONAL

A. Solve Nb x Nb eigenvalue problem

$$[\bar{\boldsymbol{K}} - \boldsymbol{\Omega}^2 \boldsymbol{I}]\ \boldsymbol{Z} = \boldsymbol{0}$$

Where $\bar{\boldsymbol{K}} = \boldsymbol{V}^T \boldsymbol{K V}$

B. Calculate orthogonal Ritz vectors $\boldsymbol{\Phi} = \boldsymbol{VZ}$

2. In the calculation of the free-vibration mode shapes the spatial distribution of the loading is completely disregarded. Therefore, many of the mode shapes which are calculated are orthogonal to the loading and do not participate in the dynamic response.

3. If dynamic loads are applied at massless degrees-of-freedom the use of all the exact mode shapes in a modesuperposition analysis will not converge to the exact solution. In addition, displacements and stresses near the application of the loads can be in significant error.

4. It is possible to calculate a set of stiffness and mass orthogonal Ritz vectors, with a minimum of computational effort, which will converge to the exact solution for any spatial distribution of loading. This approach is similar to the Lanczos method for the calculation of eigenvalues and vectors except that the starting vectors are the static displacements due to the spatial load distributions of the dynamic loading.

During the past several years it has been demonstrated that dynamic analyses based on an unique set of Load Dependent Vectors yield more accurate results than the use of the same number of exact mode shapes. The efficiency of this technique has been illustrated by solving many problems in structural response, wave propagation type problems (Bayo and Wilson, 1984) and in substructure analysis (Wilson and Bayo, 1986).

As summarized in Table 1 the starting block of vectors $\boldsymbol{u}_s$ is the static displacements obtained from an analysis using the spatial distributions of the dynamic load vectors $\boldsymbol{f}$ as input. The other vectors $\boldsymbol{V}_i$ are generated from a recurrence relationship in which the mass matrix $\boldsymbol{M}$ is multiplied by the previously obtained block of Ritz vectors $\boldsymbol{V}_{i-1}$ and used as the load vectors for a new static solution. Therefore, after the stiffness matrix $\boldsymbol{K}$ is triangularized it is only necessary to statically solve for one block of load vectors for each block of Ritz vectors. Finally, all Ritz vectors are made stiffness orthogonal.

In general, the use of load-dependent Ritz vectors is approximately six times faster than the latest subspace iteration algorithm. In addition, for most structures, the Ritz vector approach produces more accurate results than if the same number of exact eigenvectors are used.

FUNDAMENTAL EQUILIBRIUM EQUATIONS

The global dynamic equilibrium equations, at time t, of an elastic structure with nonlinear or energy dissipating elements can be written in the following form:

$$[1] \quad \boldsymbol{M\ddot{u}(t)} + \boldsymbol{C\dot{u}(t)} + \boldsymbol{Ku(t)} + \boldsymbol{R(t)}_N = \boldsymbol{R(t)}$$

Where $\boldsymbol{M}$, $\boldsymbol{C}$ and $\boldsymbol{K}$ are the mass, proportional damping and stiffness matrices, respectively. The size of these three "n x n" square matrices are equal to the number of unknown node point displacements with the nonlinear elements

removed. The time dependent vectors $\ddot{u}(t)$, $\dot{u}(t)$, $u(t)$ and $R(t)$ are the node point acceleration, velocity, displacement and external applied load, respectively. And $R(t)_N$ is the global node force vector due to the sum of the forces in the nonlinear elements and is computed by iteration at time t . The complete algorithm for the solution of structural systems with a limited number of nonlinear elements is summarized in Table 2.

The first step in the solution of Eq. (1) is to calculate a set of "m" orthogonal Ritz vectors which satisfy the following equations:

$$[2] \qquad \phi^T M \phi = I \qquad \text{and} \qquad \phi^T K \phi = \Omega^2$$

Where I is a unit matrix and Ω^2 is a diagonal matrix in which the diagonal terms are defined as $\omega^2{}_m$.

The response of the system can now be expressed in terms of these vectors by the introduction of the following matrix transformations:

$$[3] \qquad u(t) = \phi Y(t) \qquad \dot{u}(t) = \phi \dot{Y}(t) \qquad \ddot{u}(t) = \phi \ddot{Y}(t)$$

The substitution of these equations into Eq. (1) and the multiplication of both sides of the equation by ϕ^T yields a set of "m" equations of the following form:

$$[4] \qquad I\ddot{Y}(t) + \Lambda \dot{Y}(t) + \Omega^2 Y(t) = F(t) - F(t)_N$$

In which the linear and nonlinear modal forces are given by

$$[5] \qquad F(t) = \phi^T R(t) \qquad F(t)_N = \Phi^T R(t)_N$$

The assumption that the damping matrix can be diagonalized is consistent with the normal mode superposition method in which damping values are assigned, in terms of percent of critical damping, at the modal level. The diagonal terms of the Λ matrix are $-2\,\xi_m \omega_m$ in which ξ_m is the damping ratio for mode "m". It should be pointed out that the forces associated with concentrated dampers at any location in the structure can be included as part of the nonlinear force vector.

CALCULATION OF NONLINEAR FORCES

At any time t the deformations $\delta(t)$, in the local coordinate system of the nonlinear elements, can be expressed in terms of the node point displacements, $u(t)$, by a standard displacement transformation equation of the form:

$$[6] \qquad \delta(t) = A\,u(t)$$

Table 2. Summary of Nonlinear Solution Algorithm

I. INITIAL CALCULATION - PRIOR TO STEP-BY-STEP SOLUTION

A. Calculate "M" Ritz vectors $\boldsymbol{\phi}$ for the structure without the nonlinear elements. These vectors have "N" displacement degrees-of-freedom.

B. Calculate the "L" x "M" $\boldsymbol{B}$ matrix. Where "L" is the total number of degrees-of-freedom within all nonlinear elements.

C. Calculate three constants $\bar{\omega}_m$ S_m C_m for each mode to be used in constant time increment step-by-step solution.

II. NONLINEAR SOLUTION at times $\boldsymbol{\Delta t}$, $\boldsymbol{2\Delta t}$, $\boldsymbol{3\Delta t}$ --------------

A. Estimate solution at time $\boldsymbol{t}$

$$Y(t)_m = Y(t-\Delta t)_m + \Delta t \dot{Y}(t-\Delta t)_m + \frac{\Delta t}{2} \ddot{Y}(t-\Delta t)_m$$

$$\dot{Y}(t)_m = \dot{Y}(t-\Delta t)_m + \Delta t \ddot{Y}(t-\Delta t)_m$$

B. For iteration "i" calculate "L" deformations and velocities in nonlinear elements

$$\boldsymbol{\delta}(t)^i = \boldsymbol{B}\,\boldsymbol{Y}(t)^i \qquad \dot{\boldsymbol{\delta}}(t)^i = \boldsymbol{B}\,\dot{\boldsymbol{Y}}(t)^i$$

C. Based on the deformation history in nonlinear elements calculate "L" nonlinear forces $\boldsymbol{P}(t)^i$

D. Calculate new modal force vector from $\bar{\boldsymbol{F}}(t)^i = \boldsymbol{F}(t) - \boldsymbol{B}^T\boldsymbol{P}(t)^i$

E. Solve modal equations for new $\boldsymbol{Y}(t)^i$ $\dot{\boldsymbol{Y}}(t)^i$ $\ddot{\boldsymbol{Y}}(t)^i$

F. Calculate error norm
$$Err = \frac{\sum_{m=1}^{M} |\bar{f}(t)_m^i| - \sum_{m=1}^{M} |\bar{f}(t)_m^{i-1}|}{\sum_{m=1}^{M} |\bar{f}(t)_m^i|}$$

G. Check Convergence - where the tolerance Tol is specified by the user

1. If $Err > Tol$ go to II.B and continue iteration with i = i + 1

2. If $Err < Tol$ go to II.A with $\boldsymbol{t} = \boldsymbol{t} + \boldsymbol{\Delta t}$

Since $\boldsymbol{u(t)} = \boldsymbol{\Phi} \, \boldsymbol{Y(t)}$, the deformations in the nonlinear elements can be expressed directly in terms of the modal coordinate as

[7] $\boldsymbol{\delta}(t) = \boldsymbol{B}\,\boldsymbol{Y}(t)$

In which

[8] $\boldsymbol{B} = \boldsymbol{A}\,\boldsymbol{\Phi}$

It is very important to note that the $\boldsymbol{B}$ matrix is not a function of time and is normally a small matrix since its size is "the number of nonlinear deformation terms" times "the number of modes" required for the dynamic analysis. Therefore, it need be calculated only once prior to the integration of the modal equations and it can be easily retained in high-speed computer storage during all phases of the nonlinear solution.

At any time, given the deformations and history of behavior in the nonlinear elements, the forces in the nonlinear elements $\boldsymbol{P(t)}$ can be evaluated from the basic nonlinear properties and deformation history of the element. For a small number of nonlinear elements all of this information can be retained in high-speed computer memory. The nonlinear modal forces are then calculated from

[9] $\boldsymbol{F}(t)_N = \boldsymbol{B}^T \boldsymbol{P}(t)$

It is apparent that the amount of computational effort required for the calculation of these nonlinear modal forces, for each iteration, is directly proportional to the number of nonlinear degrees-of-freedom and the total number of modes squared.

SOLUTION OF NONLINEAR MODAL EQUATIONS

The algorithm for the exact solution of a modal equation, when the load varies linearly within the time increment, is summarized in Table 3. The calculation of the Load Dependent Vectors, without the nonlinear elements, is the first step prior to the solution of the modal equations. Also, the $\boldsymbol{B}$ deformation-modeshape transformation matrix need be calculated only once prior to the start of the step-by-step solution phase. A typical modal equation is of the form:

[10] $\ddot{y}(t)_m - 2\xi_m \omega_m \dot{y}(t)_m + \omega_m^{\ 2}\, y(t)_m = \bar{f}(t)_m$

Table 3. Algorithm for the Exact Solution of Modal Equation

I. MODAL EQUATIONS TO BE SOLVED AT EQUAL TIME STEPS Δt

$$\ddot{y}(\tau)_m - 2\xi_m\omega_m\dot{y}(\tau)_m + \omega_m^{\,2}\, y(\tau)_m = \bar{f}(\tau)_m$$

where $\bar{f}(\tau)_m = \bar{f}(t-\Delta t) + \dfrac{\tau}{\Delta t}(\bar{f}(t)_m - \bar{f}(t-\Delta t)_m)$

II. INITIAL CALCULATIONS

$$\bar{\omega}_m = \omega_m\sqrt{1 - \xi_m^{\,2}} \qquad \bar{\xi}_m = \xi_m\omega_m$$

$$S_m = e^{-\omega_m\xi_m\Delta t}Sin(\bar{\omega}_m\Delta t)$$

$$C_m = e^{-\omega_m\xi_m\Delta t}Cos(\bar{\omega}_m\Delta t)$$

III. STEP-BY-STEP SOLUTION PHASE

A. Given $y(t-\Delta t)_m$ and $\dot{y}(t-\Delta t)_m$ Calculate Time Increment Constants

$$b = [\,\bar{f}(t)_m - \bar{f}(t-\Delta t)_m\,]\,/\,\Delta t$$

$$A_0 = [\,\bar{f}(t-\Delta t) - 2\xi_m b/\omega_m\,]/\omega_m^2$$

$$A_1 = b/\omega_m^2$$

$$A_2 = y(t-\Delta t)_m - A_0$$

$$A_3 = [\dot{y}(t-\Delta t)_m + \bar{\xi}_m A_2 - A_1]/\bar{\omega}_m$$

$$A_4 = \bar{\omega}_m A_3 - \bar{\xi}_m A_2$$

$$A_5 = \bar{\omega}_m A_2 + \bar{\xi}_m A_3$$

B. Calculate Modal Displacement and Velocity

$$y(t)_m = A_0 + A_1\Delta t + A_2 C_m + A_3 S_m$$

$$\dot{y}(t)_m = A_1 + A_4 C_m - A_5 S_m$$

C. Increment Time, $t = t + \Delta t$, and Return to III.A

Where $\bar{f}(t)_m$ is the modal load and, for nonlinear elements, is a function of all other modal responses at the same point in time. Therefore, the modal equations must be integrated simultaneously and iteration is necessary to obtain the solution of all modal equations at time t .

The exact solution of a modal equation, for a linear variation of load within a time step, is well-known and is in terms of exponential, square root, sine and cosine functions. However, these computational intensive functions can be precalculated for all modes and used as constants for each time step and iteration as indicated in Tables 1 and 2.

NUMERICAL EXAMPLES

The examples presented in this section have been run with the SADSAP computer program (Static And Dynamic Structural Analysis Program) which has been developed by the author. The program has extensive plotting capabilities including the different components of energy within the total structural system. This allows the user to examine the effectiveness of different designs for base isolation or energy dissipation systems. Also, convergence of the method can be verified by creating plots of the input, kinetic, strain, damping, nonlinear element energy and energy error.

Example 1. Application of Ritz Vectors to Linear Dynamic Analysis

The fixed-end beam, shown in Figure 2, is subjected to a point load at the center of the beam. The load varies in time as a constant unit step function.

The maximum displacement under the load and the maximum moment at midspan are summarized in Figure 3. Since the damping ratio is selected at one percent the maximum displacement and moment occur at 0.046 seconds as shown in Table 4.

The results clearly indicate the advantages of the use of load-dependent vectors. One notes that the free-vibration modes 2, 4, 6 and 8 are not excited by the loading since they are nonsymmetrical. However, the load dependent algorithm generates only the symmetrical modes. In fact, the algorithm will fail, for this case, if more than five vectors are requested.

Both methods give good results for the maximum displacement. The results for maximum moment, however, indicate that the load-dependent vectors give significantly better results and converge from above the exact solution.

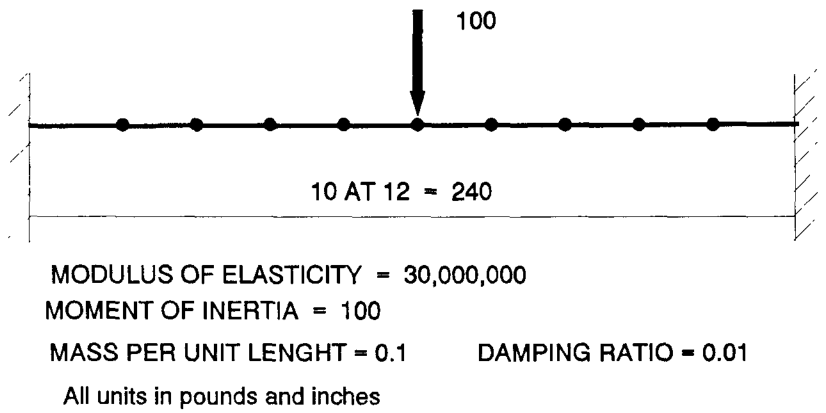


FIG. 2. Dimensions, Stiffness, Mass and Loading for Beam Structure

Table 4. Results from Dynamic Response Analysis of Beam Structure

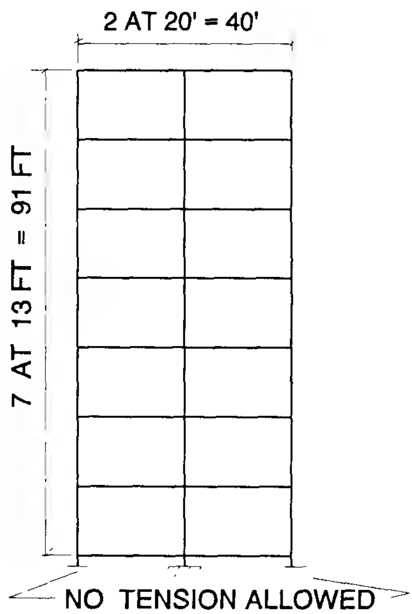
Number of Vectors	Free-Vibration Mode Shapes		Load-Dependent Ritz Vectors	
	Displacement	Moment	Displacement	Moment
1	0.004572 (-2.41)*	4178 (-22.8)	0.004726 (+0.88)	5907 (+9.2)
2	0.004572 (-2.41)	4178 (-22.8)	0.004591 (-2.00)	5563 (+2.8)
3	0.004664 (-0.46)	4946 (-8.5)	0.004689 (+0.08)	5603 (+3.5)
4	0.004664 (-0.46)	4946 (-8.5)	0.004688 (+0.06)	5507 (+1.8)
5	0.004681 (-0.08)	5188 (-4.1)	0.004685 (0.00)	5411 (0.0)
7	0.004683 (-0.04)	5304 (-2.0)		
9	0.004685 (0.00)	5411 (0.0)		

* indicates error in percent

It is clear that the use of free-vibration mode shapes is not the best vectors to be used in mode superposition dynamic response analysis. Not only is the calculation of the exact free-vibration mode shapes computationally expensive, it requires more vectors which increase the number of modal equations to be integrated and stored within the computer.

<u>Example 2. Static Nonlinear Analysis of Frame Structure</u>

In order to illustrate the use of the algorithm for the solution of a structure subjected to static loads, it is assumed that the external columns, of the seven story frame structure shown in Fig. 3, cannot take axial tension and moment at the foundation level and can uplift. The vertical foundation stiffness is 500 kips per inch at the external columns and 100 kips per inch at the center column. The dead load is 80 kips per story and is applied as concentrated vertical loads of 20 kips at the external columns and 40 kips at the center column. The static lateral load is specified as 50 percent of the dead load.



BEAMS
$I = 80{,}000\ IN^4$ $A = 300\ IN^2$

CENTER COLUMN
$I = 100{,}000\ IN^4$ $A = 400\ IN^2$

LEFT AND RIGHT COLUMNS

$I = 50{,}000\ IN^4$ $A = 200\ IN^2$

MODULUS OF ELASTICITY
$E = 4{,}000\ Kips/IN^2$

FIG. 3a. Properties of Frame Structure

The mass of the structure, for the purpose of calculating the dynamic response, is calculated directly from the dead load. The fundamental period of the structure, with the external columns not allowed to uplift, is 0.708 seconds. The fundamen-

tal period of the structure, with both external column foundations removed, is 1.691 seconds.

The static load patterns used to generate the series of Ritz vectors are shown in Fig. 3b. The first load pattern represents the mass proportional lateral earthquake load. The second pattern represents the vertical dead load. The last two load patterns represent the possible contact forces which exist at the foundation of the external columns. It is very important that equal and opposite load patterns be applied at each point where a nonlinear element exists. These vectors allow for the accurate evaluation of member forces at the contact points. For this example, the vectors will not be activated in the solution when there is uplift at the base of the columns since the axial force must be zero. Also, the total number of Ritz vectors used should be a multiple of the number of static load patterns in order that the solution is complete for all possible loadings. In addition, care should be taken to make sure that all vectors are linear independent.

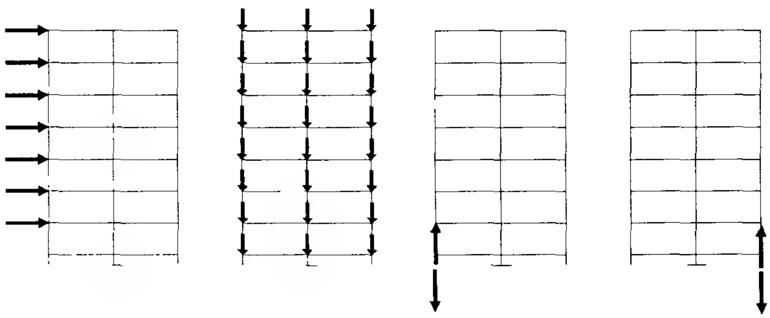


FIG. 3b. Four Static Load Vectors Used in Analysis

For this example, the dead load is applied at time zero and reaches its maximum value at one second as shown in Fig. 3c. The time increment used is 0.10 seconds. The modal damping ratios are set to 0.99 for all modes; therefore, the dynamic solution converges to the static solution in less than one second. The lateral load is applied at two seconds and reaches a maximum value at three seconds. At four seconds, after 40 load increments, a static equilibrium position is obtained as indicated in Figures 3d and 3e.

It is of interest to note that it is impossible for a structure to fail under static loads only, since at the point of collapse inertia forces must be present. Therefore, the application of static load increments with respect to time is a physically realistic approach.

DAMPING RATIO FOR ALL MODES = 0.999

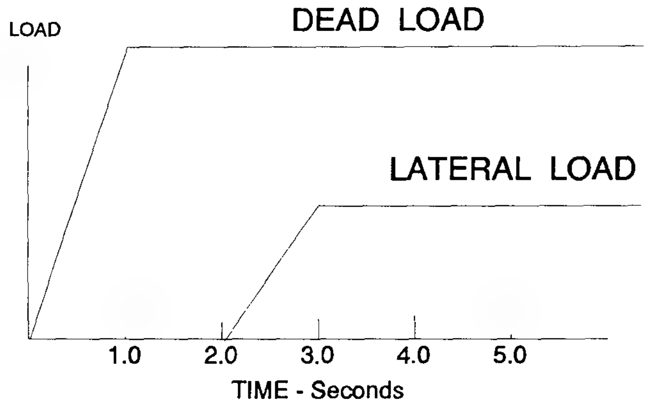


FIG. 3c. Application of Static Loads vs. Time

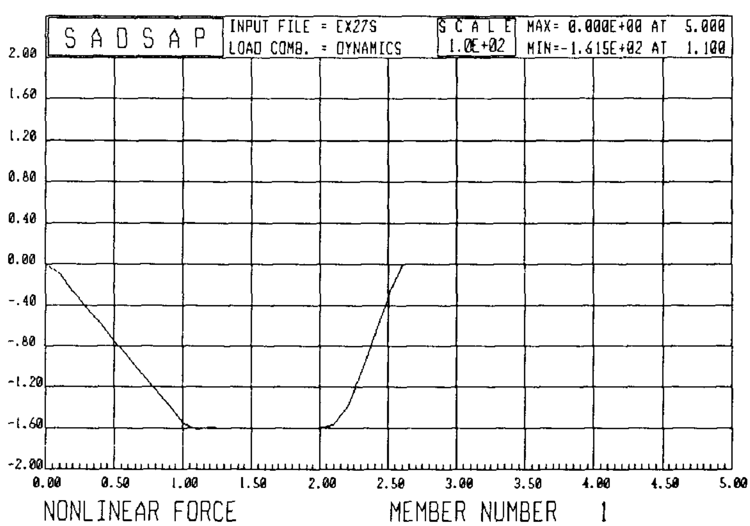


FIG. 3d. Vertical Force at Foundation of Left Column

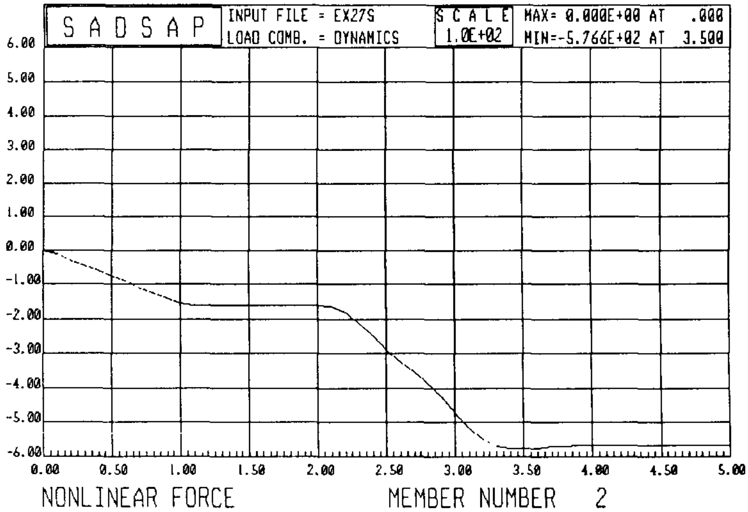


FIG. 3e. Vertical Force at Foundation of Right Column

It should be pointed out that the converged solution is the exact static solution for this problem since all possible combinations of the static vectors have been included in the analysis. The magnitude of the mass, damping and the size of the time step used will not effect the value of the converged static solution.

Example 3. Nonlinear Dynamic Analysis of Frame Structure

The same frame structure, which was defined in Example 2, is subjected to Loma Prieta Earthquake ground motions recorded on the East side of the San Francisco Bay and has a maximum acceleration of 20.1 percent and a maximum ground displacement of 5.81 inches. The portion of the acceleration record used is shown in Fig. 4a and has been corrected to zero acceleration, velocity and displacement at the end of the record.

The dead load was applied as a ramp function in the time interval zero to one second. The lateral earthquake load is applied starting at two seconds. Eight Ritz vectors and a modal damping value of five percent were used in the analysis. The axial force at the left foundation vs. time is shown in Fig. 4b.

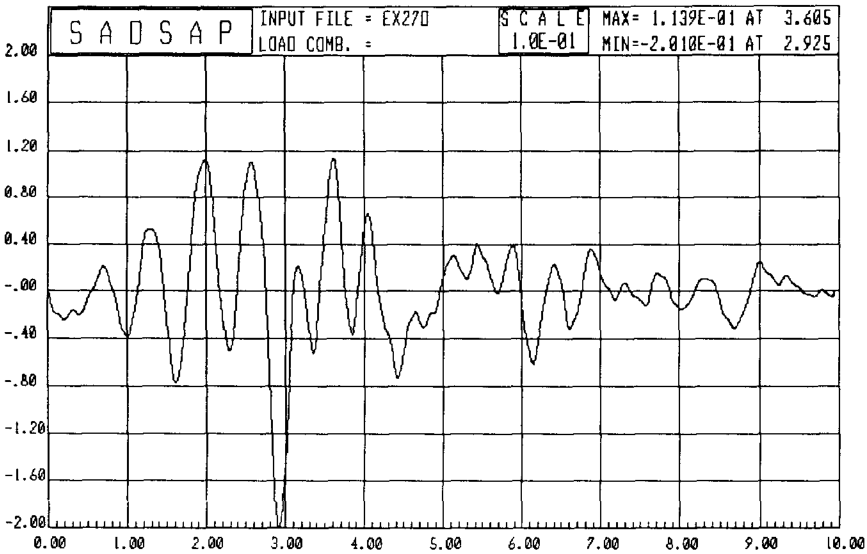


FIG. 4a. Segment of Loma Prieta Acceleration Record

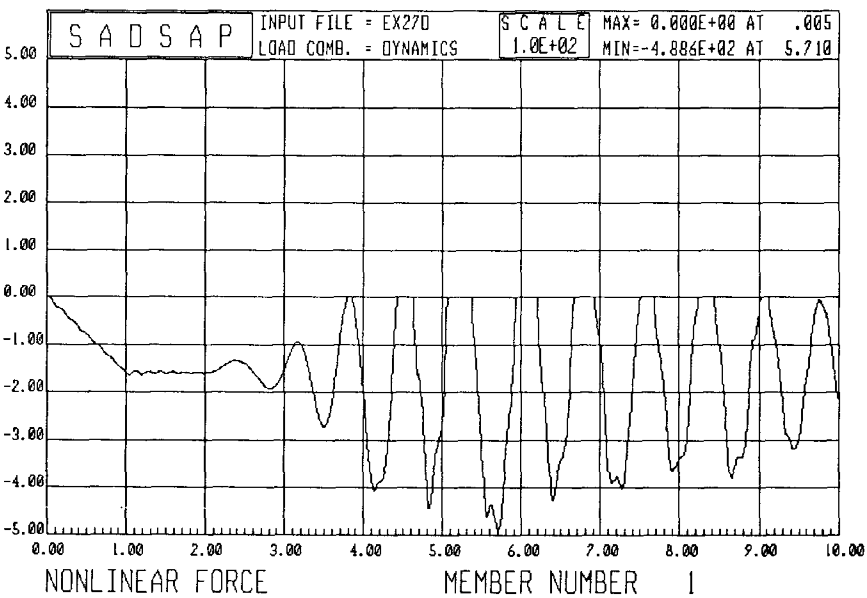


FIG. 4b. Vertical Force at Foundation of Left Column

It is of considerable interest to compare the behavior of the building which is not allowed to uplift with the behavior of the same building which is allowed to uplift. These results are summarized in Table 5.

Table 5. Comparison of Building Analysis With and Without Uplifting

MAXIMUM VALUE	WITHOUT UPLIFTING	WITH UPLIFTING	PERCENT DIFFERENCE
Top Displacement	3.88 Inches	3.99 Inches	+2.8 %
Axial Column Force	455 Kips	489 Kips	+7.5%
Base Shear	247 Kips	196 Kips	-20.6%
Base Moment	212,000 k-in	153,000 k-in	-27.8%
Strain Energy	442 k-in	303 k-in	-31.4%
Computational Time	47 seconds	90 seconds	+92%

As one would expect, the lateral displacement at the top of the structure increases by less than three percent. Also, the axial forces in the external base columns increase by approximately eight percent. However, allowing column uplifting reduces significantly the base shear, overturning moment and strain energy stored in the structure. It is apparent, for this structure, that uplifting is a "natural" base isolation system. This reduction of forces in a structure due to uplifting has also been observed in shaking table tests (Clough and Huckelbridge, 1977). However, it has not been used extensively for real structures because of the lack of precedent and the inability of the design engineer to easily compute the dynamic behavior of an uplifting structure.

For this small example there appears to be a large increase in computational time; however, for a large structural system, in which the time for the formation and solution of the equilibrium equations and the calculation of mode shapes is large, the additional time to integrate the nonlinear modal equation is often less than five percent of the total computational time.

Example 4. Seismic Analysis of Elevated Water Tank

A nonlinear earthquake response analysis of an elevated water tank was conducted using a well-known commercial computer program in which the stiffness matrix for the complete structure was recalculated for each time step and equilibrium was obtained using iteration. The structural system and analysis had the following properties:

92	nodes with 236 unknown displacements
103	elastic frame elements
56	nonlinear diagonal brace elements - tension only
600	time steps at 0.02 seconds

The solution time on two different computers are listed below:

Intel 486	3 days	4320	minutes
Cray XMP-1	3 hours	180	minutes

The same structure was solved by SADSAP using the numerical method presented in this paper and an Intel 486 in less than 3 minutes.

FINAL REMARKS

It is now common practice in civil engineering design to restrict the nonlinear behavior to a small number of predefined locations within a structure. In this paper an efficient computational method has been presented to perform the dynamic response analysis of these types of structural systems.

A standard time-history analysis is performed by the mode superposition method. The modal equations are integrated by a method which is exact for a linear variation of the load during the defined time increment. If nonlinear elements are present the loads in the nonlinear elements are calculated at the end of each time step. The element loads are converted to modal loads and applied to the right-hand- side of the modal equations. Iteration is then performed within the time step until the loads at the end of the time increment converge.

In order for the method, which is presented here, to work the structural system must be structurally stable without the nonlinear elements. Since all structures can be made to satisfy this condition if linear elements with very small stiffness are added to the basic computer model, this is not a real limitation of the method. If a structural system satisfies these conditions it is possible to calculate the nonlinear dynamic response with almost the same numerical effort as required by a linear dynamic analysis.

It should be noted that structures subjected to static loads can also be solved by the method. It is only necessary to apply the loads very slowly to a constant value and add a small modal damping value. Therefore, the final converged solution will be in static equilibrium and will not contain inertia forces. It should be pointed out that it is necessary to use Load Dependent Vectors, and not the dynamic mode shapes, if static problems are to be solved by this approach.

The SADSAP computer program calculates and plots, as a function of time, the total input energy, strain energy, kinetic energy and the dissipation of energy by modal damping and nonlinear elements. In addition, an energy error is calculated which allows the uses to evaluate the appropriate time step size. Also, the energy calculation option allows different structural designs to be compared. In many cases a good design, for a specified dynamic loading, is one which has a minimum amount of strain energy absorbed within the structural system.

As in the case of normal linear mode superposition analysis, it is the responsibility of the user to check, with multiple analyses, that a sufficiently small time step and the appropriate number of modes are used. This approach will assure that the method will converge to the exact solution.

Using the numerical methods presented in this paper the computational time required for a nonlinear dynamic analysis of a large structure, with a small number on nonlinear elements, is only a few percent more than the computational time required for a linear dynamic analysis of the same structure.

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ANALYTICAL MODELING OF IMPACT ON REINFORCED CONCRETE STRUCTURES

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ABSTRACT

Analytical models are presented for direct impact between a mass and a structure. The finite element-based model includes both elastic and elasto-viscoplastic material properties. Two separate procedures for detecting and modeling contact and release conditions are discussed. The application of the model is illustrated by results for the analysis of impact between two elastic rods, impact between a moving mass and a reinforced concrete bridge barrier, a sphere striking a rod, and a Split Hopkinson Bar test of a reinforcing bar embedded in concrete.

INTRODUCTION

Civil engineering structures may be subjected to severe loading due to impact. Examples include transportation structures subjected to impact of vehicles, marine and offshore structures subjected to ice impact, impact due to projectiles as a result of explosions, and impact due to falling objects caused by earthquake or accident. Design for such impact forces is often empirical in nature because impact phenomena are not well understood from the viewpoint of structural design and there is considerable uncertainty about the load levels and loading rates that can be expected during an impact event, as well as uncertainty about the response of structures to high loading rates. Analytical procedures are needed to improve our understanding of behavior of structures subjected to impact and to provide a basis for more rational design procedures than are currently available.

This paper outlines two approaches to analytical modeling of impact using the finite element method and nonlinear material response characteristics. In the first approach (Riad, 1991), potential contact nodes are identified in the colliding bodies and the contact and force conditions at these potential contact nodes are

monitored during the contact/impact event. In the second approach (Maksoud, 1991), a contact element is inserted between potential contact nodes and the contact is monitored in terms of the force developed in the contact element.

Since impact often involves severe loading conditions, nonlinear material response must be considered. For this study, elasto-viscoplastic material behavior is assumed using the formulation presented by Owen and Hinton (1980). The impact formulations are implemented in the computer program DYNPAK developed by Owen and Hinton. The formulation of the analytical procedures along with several example solutions are presented in the following sections.

EQUATIONS OF MOTION FOR COLLIDING BODIES

The equation of motion of a structural system under dynamic loading at time station t_n is written as

[1] $$[M]\{\ddot{u}\}_n + [C]\{\dot{u}\}_n + \{p\}_n = \{f\}_n$$

where [M] and [C] are the global mass and damping matrices, respectively, {p} is the global vector of internal resisting nodal forces, {f} is the vector of consistent nodal forces for the applied body and surface traction forces grouped together, $\{\ddot{u}\}$ is the global vector of nodal accelerations, and $\{\dot{u}\}$ is the global vector of nodal velocities. For the class of impact problems considered, the mass matrix is diagonal and damping can be neglected. As a result, the damping matrix [C] is zero and the equation of motion at station t_n reduces to

[2] $$[M]\{\ddot{u}\}_n + \{p\}_n = \{f\}_n$$

Fig. 1 shows schematically a mass about to strike a vertical cantilever beam. The structure and impacting mass are subdivided into finite elements whose nodes are represented as either contact or non-contact nodes. Node groups a and d are non-contact nodes and groups b and c are contact nodes. During contact, equal and opposite nodal forces (τ) will develop between the adjacent contact nodes. Assuming no other external forces are present, the equations of motion during contact for the mass can be written as

[3] $$\begin{bmatrix} M_a & 0 \\ 0 & M_b \end{bmatrix} \begin{Bmatrix} \ddot{u}_a \\ \ddot{u}_b \end{Bmatrix} + \begin{bmatrix} K_{aa} & K_{ab} \\ K_{ba} & K_{bb} \end{bmatrix} \begin{Bmatrix} u_a \\ u_b \end{Bmatrix} + \begin{Bmatrix} 0 \\ -\tau_b \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \end{Bmatrix}$$

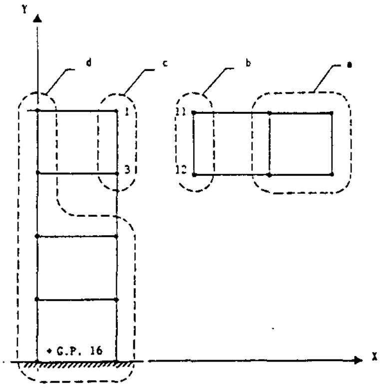


FIG. 1. Finite Element Model for Structure and Impacting Mass

For the structure, the equations of motion during impact are given by

$$[4] \qquad \begin{bmatrix} M_c & 0 \\ 0 & M_d \end{bmatrix} \begin{Bmatrix} \ddot{u}_c \\ \ddot{u}_d \end{Bmatrix} + \begin{bmatrix} K_{cc} & K_{cd} \\ K_{dc} & K_{dd} \end{bmatrix} \begin{Bmatrix} u_c \\ u_d \end{Bmatrix} + \begin{Bmatrix} \tau_c \\ 0 \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \end{Bmatrix}$$

In these expressions, M and K are submatrices for the lumped masses, and stiffness coefficients, respectively, for the different nodal subgroups a through d; τ is the vector of contact forces; $\ddot{u}$ is the vector of nodal accelerations and u is the vector of nodal displacements. Note that during contact

$$u_b = u_c,\ \ddot{u}_b = \ddot{u}_c \text{ and } \tau_c = -\tau_b.$$

Solving expressions (3) and (4) above gives the equation of motion of the combined system during impact

$$[5] \qquad \begin{bmatrix} M_a & 0 & 0 \\ 0 & (M_b+M_c) & 0 \\ 0 & 0 & M_d \end{bmatrix} \begin{Bmatrix} \ddot{u}_a \\ \ddot{u}_c \\ \ddot{u}_d \end{Bmatrix} + \begin{bmatrix} K_{aa} & K_{ab} & 0 \\ K_{ba} & (K_{bb}+K_{cc}) & K_{cd} \\ 0 & K_{dc} & K_{dd} \end{bmatrix} \begin{Bmatrix} u_a \\ u_c \\ u_d \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \\ 0 \end{Bmatrix}$$

The central difference method is used to solve the equations of motion. At each degree of freedom i for the structure nodes a, mass nodes d or the coupled node (b+c) at any time station t_n during contact, expression (5) is written as

[6] $$\{m_i\} \left[\frac{u_{n+1} - 2\, u_n + u_{n-1}}{(\Delta t)^2}\right]_i + \{p_i\}_n = \{0\}$$

which is solved for the unknown displacement u_{n+1} at the next time station t_{n+1}.

During contact, the contact force can be obtained by rearranging Eq. (4), i.e.,

[7] $$\begin{Bmatrix} \tau_c \\ 0 \end{Bmatrix} = - \begin{bmatrix} M_c & 0 \\ 0 & M_d \end{bmatrix} \begin{Bmatrix} \ddot{u}_c \\ \ddot{u}_d \end{Bmatrix} - \begin{bmatrix} K_{cc} & K_{cd} \\ K_{dc} & K_{dd} \end{bmatrix} \begin{Bmatrix} u_c \\ u_d \end{Bmatrix}$$

From the first equation of expression (7) and using the central difference method, the contact force for a typical time step t_n at each contact degree of freedom j is computed as

[8] $$\{\tau_c\}_j = - \{m_c\}_j \left[\frac{u_{n+1} - 2\, u_n + u_{n-1}}{\Delta t^2}\right]_j - \{p_c\}_n$$

At each time step, a check is made of the contact conditions to determine whether a change in contact has occurred. As long as $\{\tau_c\}_j$ as computed from [8] is positive, contact is still in effect. Once this contact force becomes negative, contact is no longer taking place and release conditions have occurred.

Contact and Release Conditions

The analysis procedure for evaluating the conditions at contact and release follows the general approach given by Hughes et al. (1976).

The first occurrence of contact, at any given pair of contact nodes i, of the system is determined by monitoring displacements at potential contact nodes and computing compressive nodal contact forces $\tau_+ > 0$ normal to the interface at node i using

[9] $$(\tau_i)_+ = (\tau_i) - \frac{m_{bi}\, m_{ci}\, \left((\ddot{u}_{bi})_- - (\ddot{u}_{ci})\right)}{(m_{bi} + m_{ci})}$$

where subscripts (-) and (+) refer to the conditions before and after impact, respectively, at the current time station; m_{bi}, m_{ci} and $\ddot{u}_{bi}$, $\ddot{u}_{ci}$ are the vectors of nodal masses and accelerations before contact for adjacent contact nodes in subgroups b and c, respectively. Once contact has occurred, the common post-impact accelerations and velocities for the nodes in contact are given, respectively, by

$$[10] \qquad (\ddot{u}_i)_+ = \frac{m_{bi}(\ddot{u}_{bi})_- + m_{ci}(\ddot{u}_{ci})}{m_{bi} + m_{ci}}$$

$$[11] \qquad (\dot{u}_i)_+ = \frac{m_{bi}(\dot{u}_{bi})_{-1} + m_{ci}(\dot{u}_{ci})_{-1}}{(m_{bi} + m_{ci})}$$

where $(\dot{u}_{bi})_{-1}$ and $(\dot{u}_{ci})_{-1}$ are the vectors of nodal velocities before impact and the subscript (-1) refers to the value at the previous time station.

The algorithm starts at time station $t_n = 0$ when the structure is initially at rest and the mass is moving at velocity v_o. The initial displacement u_o for both the structure as well as the mass is zero. Obviously, no contact has taken place yet, i.e., $\tau_o = 0$. The kinematic conditions of the displacements, velocities, and accelerations at time station $t_n = 0$ are now examined.

<u>Kinematic Conditions at Time Station $t_n = 0$</u>

The mass has displaced by a distance u_1 from its original position given by

$$[12] \qquad u_1 = \Delta t(v_o)$$

The initial position coordinates of the contact nodes of the structure and impacting mass are specified to be the same. Based on the assumption that no penetration is allowed, an incompatibility of the displacements of the structure and mass has occurred at the interface. To correct this incompatibility, the same displacement u_1 is applied to the structure such that

$$[13] \qquad (u_1)_b = (u_1)_c = \Delta t(v_o)$$

As for the velocities, the mass is initially moving at constant velocity v_o and the structure is initially at rest. The initial velocities of the mass and structure, respectively, are described by

$$[14] \qquad (\dot{u}_o)_b = v_o$$

$$[15] \qquad (\dot{u}_o)_c = 0$$

The accelerations of the structure and impacting mass can be expressed, respectively, as

[16] $$(\ddot{u}_o)_b = 2\left[\frac{(u_1)_b - (u_o)_b - \Delta t(v_o)_b}{(\Delta t)^2}\right] = 0$$

[17] $$(\ddot{u}_o)_c = 2\left[\frac{(u_1)_c - (u_o)_c - \Delta t(v_o)_c}{(\Delta t)^2}\right] = \frac{2u_1}{(\Delta t)^2}$$

At the end of this time step, i.e., at time station $t_n = 1$, the contact nodes are coalescing over each other. The difference of displacements d is zero, more precisely

[18] $$d = (u_1)_b - (u_1)_c = 0 < TOL$$

where TOL is a selected tolerance criterion. A value of TOL $= 10^{-10}$L, where L is a problem characteristic length, was found to be adequate. Although the tolerance criterion is satisfied, the contact forces $\{\tau_0\}_+$ are negative; therefore, contact has not occurred yet. Proceed to the next step, i.e., time station $t_n = 1$, and compute the contact force $\{\tau_1\}_+$ after impact as a function of the acceleration $\{\ddot{u}_1\}_-$ before impact computed at time station $t_n = 1$.

<u>Kinematic Conditions at Time Station $t_n = 1$</u>

The contact force $\{\tau_1\}_+$ is computed as a function of the accelerations before impact as

[19] $$(\tau_1)_+ = (\tau_1)_- - \frac{(m_b)(m_c)((\ddot{u}_b)_- - (\ddot{u}_c)\ )}{m_b - m_c}$$

Once a contact force $(\tau_1)_+ > 0$ has been developed, contact is assumed to have occurred and the common post-impact accelerations and velocities of the nodes in contact are given, respectively, by

[20] $$(\ddot{u}_1)_+ = \frac{(m_b)(\ddot{u}_b\ )_1 + (m_c)(\ddot{u}_{c-})_1}{(m_b + m_c)}$$

[21] $$(\dot{u}_1)_+ = \frac{(m_b)(\dot{u}_{b-})_0 + (m_c)(\dot{u}_c\ )_0}{(m_b + m_c)}$$

These accelerations and velocities are used to correct the displacements computed earlier without regard to the possibility of occurrence of contact.

Corrections for the Displacements to Account for Occurrence of Contact

Expression (21) for the velocity can be rewritten using the central difference approximation as

[22]
$$(\dot{u})_1 = \left[\frac{u_{2\ corr.} - u_o}{2\ \Delta t} \right]$$

which is solved for the corrected displacement $u_{2\ corr.}$ to give

[23]
$$u_{2\ corr.} = 2\ \Delta t\ (\dot{u})_1$$

Similarly, expression (20) for the acceleration is rewritten as

[24]
$$(\ddot{u})_1 = \left[\frac{u_{2\ corr.} - 2\ u_{1\ corr.} + u_o}{(\Delta t)^2} \right]$$

which is solved for $u_{1\ corr.}$ giving

[25]
$$u_{1\ corr.} = 0.5\ (\ddot{u})_1\ \left[u_{2\ corr.} - \Delta t^2 (\ddot{u})_1 + u_o \right]$$

The difference in displacements d is computed using the corrected values for the displacements and is checked against the chosen tolerance TOL. If $d < TOL$ together with a positive contact force, contact is sure to have occurred. After proceeding to the next time step the conditions for a typical impact are examined, as explained previously.

Release Conditions During a Typical Time Step

Assuming that at time station $t_n = p$, i.e. at time step p+1, the contact force $\{\tau_c\}_p$ became negative. At that time station, the displacements, velocities and accelerations at each contact degree of freedom i were computed without regard to the occurrence of release. The release conditions can be viewed as being the same as impact conditions with the time frame running backwards. Details of the formulation are given in Riad (1991).

Implementation in Computer Program

The impact formulation described above was implemented in the computer program DYNPAK presented in the text by Owen and Hinton (1980). The program incorporates plane stress, plane strain, and axisymmetric elements. The equations of motion are solved at successive time steps by the explicit central difference scheme, and the program contains an elasto-viscoplastic material model as summarized briefly in the following section.

ELASTO-VISCOPLASTIC MATERIAL MODEL

The elasto-viscoplastic material model provides an adequate representation of certain aspects of nonlinear response of civil engineering materials such as concrete and reinforcing steel. The model assumes an initial elastic response followed by plastic straining which is expressed in terms of the rate of visco-plastic straining. The visco-plastic strain rate can be expressed as,

(26) $$\dot{\epsilon}_{vp} = \gamma < \phi(F) > \frac{\partial f}{\partial \sigma}$$

where γ is the fluidity parameter controlling the plastic flow rate, $f = f(\sigma, \dot{\epsilon}_{vp}, k)$ is a plastic potential, $\frac{\partial f}{\partial \sigma}$ is a vector normal to the yield surface and defines the direction of viscoplastic flow and the notation $< >$ implies

[27] $$< \phi(F) > = \begin{matrix} 0 & for\ F \leq 0 \\ \phi(F) & for\ F > 0 \end{matrix}$$

The flow function ϕ(F) is defined as

[28] $$\phi(F) = \frac{F}{\sigma_o} = \frac{\{f(\sigma) - \sigma_o\}}{\sigma_o}$$

where f(σ) depends on the state of stress only.

To define the elasto-plastic material model it is necessary to define the plastic potential function, f(σ) and the fluidity parameter, γ. Following associated plasticity theory the plastic potential can be replaced by a yield function. Yield functions available in the program include von Mises, Tresca, Mohr-Coulomb, and Drucker-Prager. The fluidity parameter (inverse of viscosity parameter) determines the rate at which plastic strains develop. Some guidance is available in the work of Bicanic and Zienkiewicz (1983), based on analysis of dynamic test data, for selection of an appropriate fluidity parameter. They present the following expressions for concrete and steel:

Concrete: $\gamma_c = 10^a(\dot{\epsilon}_e)^b$ Steel: $\gamma_s = a(\dot{\epsilon}_s)^b$

Typical values of the parameters a and b as presented by Bicanic and Zienkiewicz are:

Concrete: $a = -2.93, b = 0.76$ Steel: $a = 1.54, b = 0.97$

Full details of the elasto-viscoplastic formulation are given by Owen and Hinton.

CRITICAL TIME STEP AND STABILITY OF SOLUTION

A key decision in a time-stepping solution of the equations of motion is the selection of a suitable time step. For an economical solution the largest feasible time step is required. However the time step must also be small enough to ensure stability and accuracy of the solution. For wave propagation problems of the type involved in impact, the time taken for a stress wave to cross an element is a major factor in determining the time step. For uniaxial problems the critical time step is given by,

[29] $$\Delta t = \frac{L_e}{c}$$

where L_e is the length of the smallest element and c is the wave velocity. In the examples that follow, representing the case of a uniaxial elastic rod subjected to impact, it will be shown that a time step exactly on the stability limit gives the best solution for the contact force. For two-dimensional problems and nonlinear material properties the situation is more complicated and resort was made to trial and error to obtain a suitable time step. This will be discussed further in the example problems. For additional information on the stability of time stepping solutions the reader is referred to Bathe (1982).

ALTERNATIVE FORMULATION USING CONTACT ELEMENTS

The above procedure requires that the global equations of dynamic equilibrium be combined for nodes that are in contact. An alternative approach used by Maksoud (1991) for axisymmetric problems involves the use of imaginary contact elements placed between the potential contact nodes. When the nodes are not in contact, the contact element has zero force. When the nodes come in contact, a compressive force is developed in the contact element.

The contact element is shown in Fig. 2 for a general angle of orientation ϕ.

U_1 = axial displacement of node 1

U_2 = axial displacement of node 2

g = gap in the element

Then, the interference, I, between the nodes is given by

[30] $$I = U_1 - U_2 - g$$

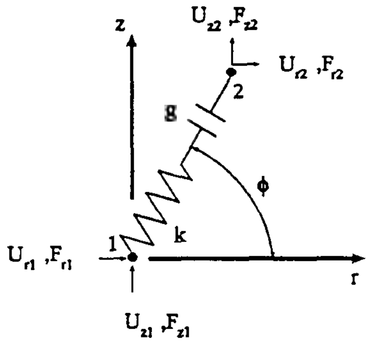


FIG. 2. Contact Element

As long as the interference is less than zero, the compression element will not develop any force. When the interference between the nodes is positive, a force is developed in the spring. If the spring has a high stiffness, the contact element, in effect, simulates a rigid link between the nodes ensuring the no interpenetration condition between the nodal points.

For the contact element with general orientation shown in Fig. 2, the interference between nodes 1 and 2 is

[31] $$I = \left(U_{r1} - U_{r2}\right) \cdot \cos(\phi) + \left(U_{z1} - U_{z2}\right) \cdot \sin(\phi) - g$$

The internal force (assuming compression is positive) in the element is

[32] $$\begin{aligned} F_o &= 0 \qquad (I \leq)) \\ &= Ik \qquad (I > 0) \end{aligned}$$

The contact element formulation was implemented by Maksoud in the computer program DYNPAK for axisymmetric problems where the orientation of the contact element is parallel to the axis of symmetry. At each time step the interference for each contact element is calculated. If the interference is negative, the element is skipped. If the interference is positive, the corresponding nodal forces are added to the force vector.

APPLICATIONS BASED ON LUMPED NODAL MASS FORMULATION

Direct Impact Between Two Elastic Rods

The solution for impact between two elastic rods is available in closed form and provides a suitable problem to verify that the numerical solution properly

models direct impact events. The results for various selected time steps provide useful insights into the effect of time step on the accuracy of solution. This problem was also used by Hughes et al. to illustrate their impact solution.

Details of the two impacting rods are given in Fig. 3. One rod is at rest and the other rod is approaching at a velocity of 2.54 mm/sec (0.1 in/sec). The critical time step is calculated as $\Delta t = 0.005$ sec.

Results are presented in Figs. 4 and 5. The crispness of the computed impact force is attributed to the selection of a time step right on the stability limit. The same conclusion was drawn by Hughes et al. The displacement of node 14 shows an increasing vibration amplitude about the contact value after release.

A second analysis was conducted with a time step of 0.0008 sec, i.e. less than the critical time step. The resulting impact force shown in Fig. 6 indicates an initial overshoot of the correct contact force but the solution stabilizes at the correct value. The displacement-time history shown in Fig. 7 is stable over the full duration of the analysis.

The results for this simple problem confirm that the solution procedure is able to model direct impact events and that the critical time step for uniaxial elastic problems is the time taken for a stress wave to cross an element.

<u>Impact of Mass on Reinforced Concrete Bridge Barrier</u>

A set of exploratory analyses was performed to evaluate the potential of the analytical procedure to model response of reinforced concrete bridge barriers subjected to vehicle impact. Data are available for a set of barrier specimens tested using a pendulum impact facility (Scanlon et al., 1990). Details of the test specimens are shown in Fig. 8. The pendulum used in the tests consists of steel element interspersed with layers of plywood to adjust the hardness of impact. Typical measured load-time histories are shown in Fig. 9 indicating a series of three force peaks over a load duration of about 80 ms.

Analytical results are presented for three finite element models of unit thickness, as follows:

BARRIER A: Elastic structure and mass

BARRIER B: Cracked Elasto-viscoplastic structure, Modified Elastic Mass, 0.01 E

BARRIER C: Cracked Elasto-viscoplastic structure, Modified Elastic Mass, 0.001 E

The finite element models are shown in Fig. 10. BARRIERS B and C also contained soft elements at the fixed support to model the flexibility of anchor bolts

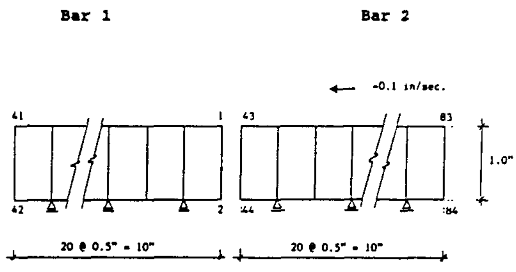


FIG. 3. Arrangement of Two Elastic Rods

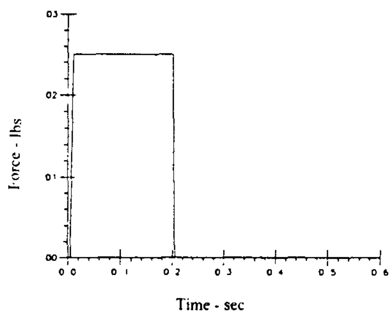


FIG. 4. Two Elastic Rods: Contact Force History, $\Delta t = 0.005$ sec

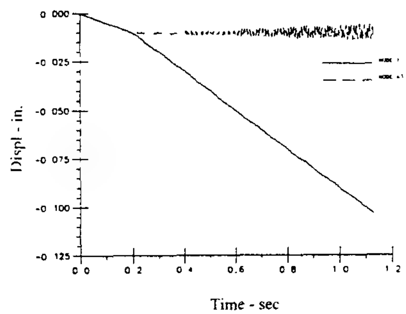


FIG. 5. Two Elastic Rods: Displacement History, $\Delta t = 0.005$ sec

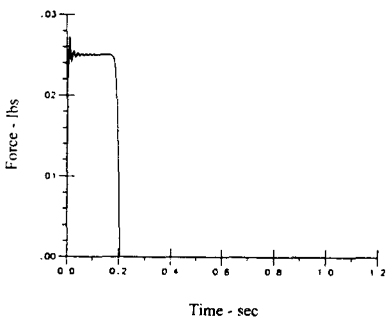


FIG. 6. Two Elastic Rods: Contact Force History, $\Delta t = 0.0008$ sec

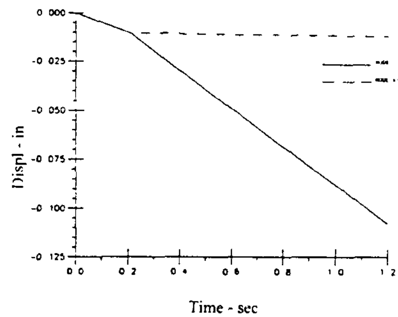


FIG. 7. Two Elastic Rods: Displacement History, $\Delta t = 0.0008$ sec

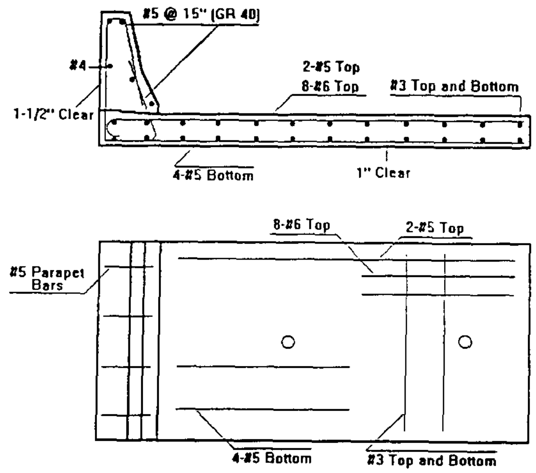


FIG. 8. Details of Barrier Test Specimens

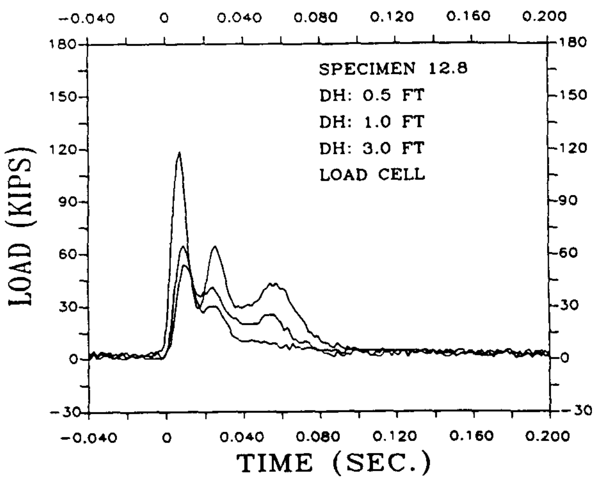
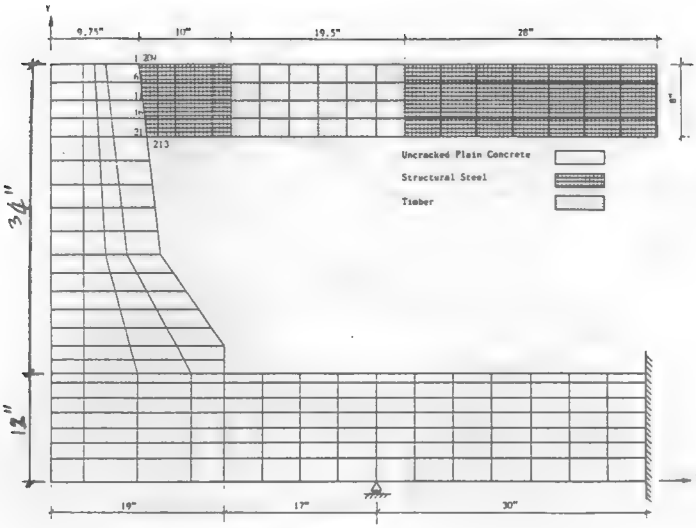
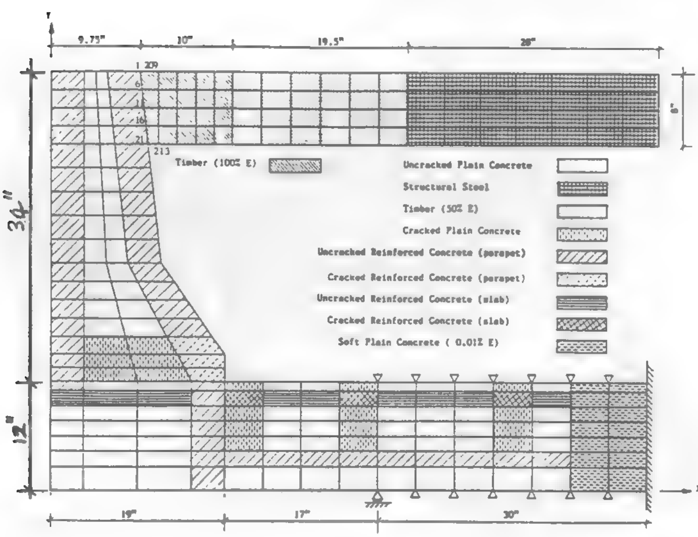


FIG. 9. Typical Impact Force-Time Histories for Bridge Barrier Specimens



(a) BARRIER A



(b) BARRIERS B and C

FIG. 10. Finite Element Models for Barrier Analysis

used to tie down the test specimens to the support. The contact force history for BARRIER A, given in Fig. 11, shows a series of force spikes over a time period of about 18 ms. The corresponding displacement-time history is shown in Fig. 12. The response clearly does not bear much resemblance to the actual behavior measured in tests as presented in Fig. 9. This can be attributed to two main factors. Firstly, the concrete specimens exhibited cracking at fairly low impact levels, and, secondly, the pendulum mass is not a solid entity but consists of several components with a certain amount of "slop" that contributes to energy dissipation in the system. In addition, rigid body movements of the specimens were recorded indicating some flexibility in the anchoring arrangement to the foundation. Therefore, modifications were made to BARRIERS B and C to account for these factors.

These modifications included the introduction of cracked concrete zones, soft elements at the fixed end to allow for anchor bolt flexibility, and reduction in stiffness in the impacting mass.

Results for BARRIER B, given in Fig. 13, show continuous contact for the first 17 ms followed by several spikes up to 67 ms when the run was terminated. The displacement history at the top contact nodes shown in Fig. 14 illustrates the separation of the top nodes after the initial continuous contact. The displacements are of the same order of magnitude as measured in the tests.

To increase the duration of continuous contact, the elastic stiffness of the mass was reduced further by a factor of 10. The resulting force-time history is shown in Fig. 15, indicating continuous contact for a duration of 80 ms when the run was terminated. The displacement-time history, Fig. 16, also shows continuous contact with a peak displacement similar to that for BARRIER B.

While the modeling used for the barriers is somewhat crude, the results show promise for accurate modeling of impact events on reinforced concrete structures.

APPLICATIONS BASED ON CONTACT ELEMENT FORMULATION

Sphere Striking Concrete Rod

The contact element and axisymmetric formulation was applied to the case of a steel sphere striking a concrete rod for which experimental data were reported by Goldsmith et al. (1966). Details of the sphere-rod system are shown in Fig. 17. and the finite element model is illustrated in Fig. 18. Results of elastic analysis, shown in Fig. 19, indicate the force on the contact element as a series of spikes. However, the computed axial strain is a smooth function of time (Fig. 20).

Examples of computed strains for the elasto-viscoplastic case are shown in Fig. 21 for a range of assumed fluidity parameters. Reasonable correlation between computed and measured strains can be seen for Case 7.

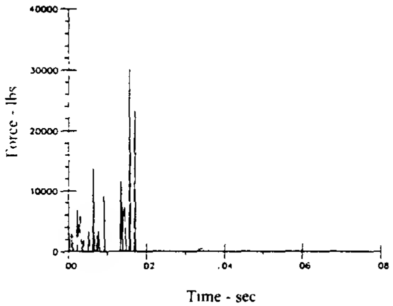


FIG. 11. Force-Time History, BARRIER A

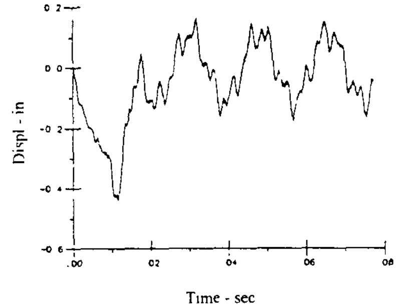


FIG. 12. Displacement-Time History, BARRIER A

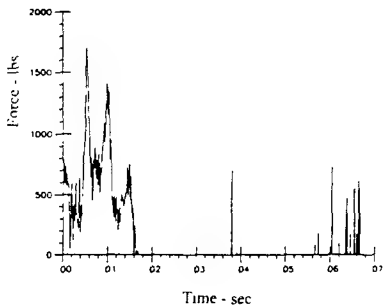


FIG. 13. Force-Time History, BARRIER B

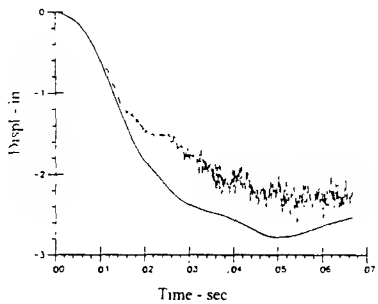


FIG. 14. Displacement-Time History, BARRIER B

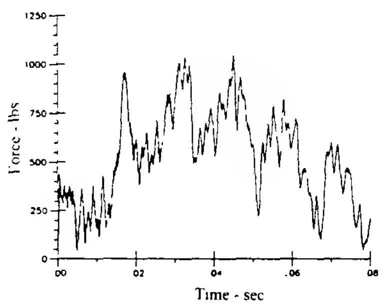


FIG. 15. Force-Time History, BARRIER C

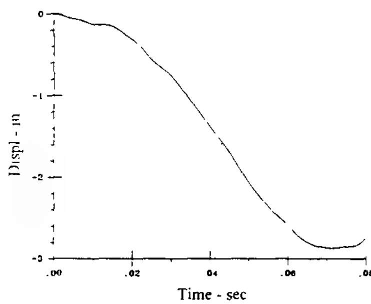


FIG. 16. Displacement-Time History, BARRIER C

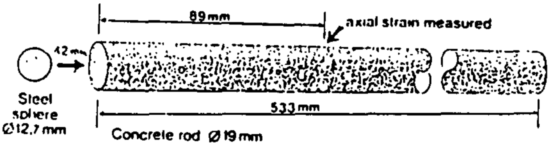


FIG. 17. Details of Sphere Striking Concrete Rod

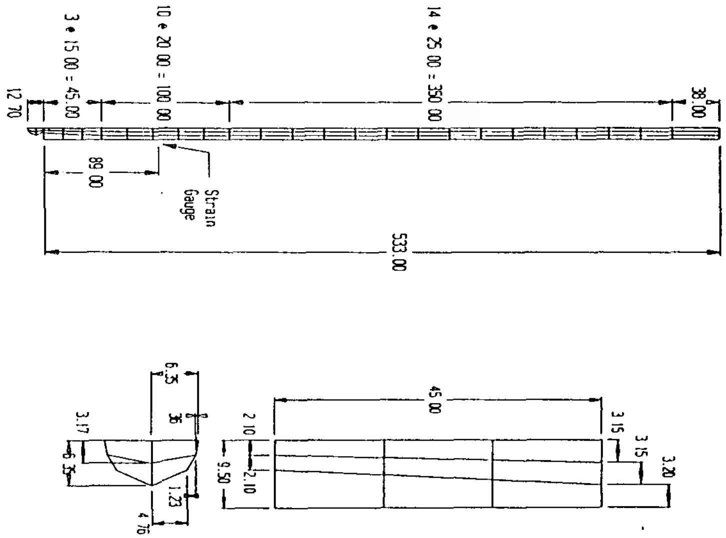


FIG. 18. Finite Element Model for Sphere/Rod

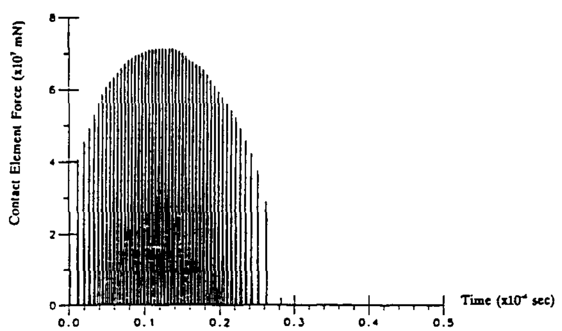


FIG. 19. Contact Element Force

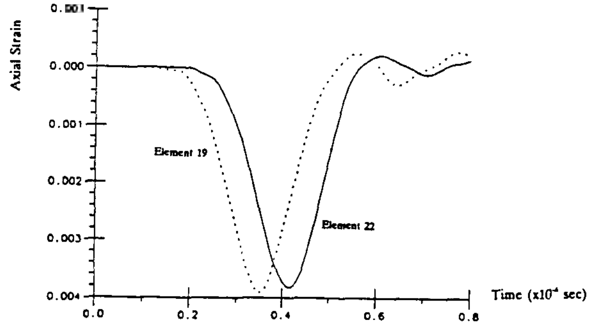
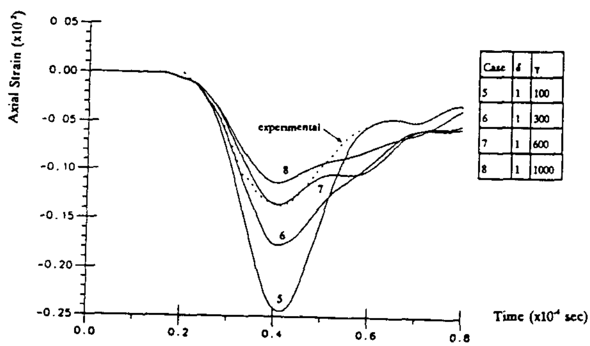


FIG. 20. Axial Strain in Rod: Elastic Analysis



Case	δ	γ
5	1	100
6	1	300
7	1	600
8	1	1000

FIG. 21. Strains in Rod: Viscoplastic Analysis

Reinforcing Bar Embedded in Concrete

Vos et al. (1981) reported results of a Split Hopkinson Bar test of a reinforcing bar embedded in a cylindrical concrete specimen. The finite element discretization of the system is shown in Fig. 22. The impact force is delivered by the steel mass striking the rubber layer on the anvil. Because no details of the rubber layer were provided, the modulus of elasticity for the rubber layer was assumed as E_s/r with values for r ranging from 1 to 10,000 to determine the sensitivity of the solution to this value. A simple tensile strength criterion was used to model cracking in the concrete elements using tensile strength values of f = 1, 2, 3 and 4 MPa. Typical results are shown in Figs. 23, 24, and 25 for displacement at the end of the steel reinforcing bar, shear stress in the bar, and crack patterns n the concrete. In general, the results were found to be somewhat sensitive to assumed values of stiffness of the rubber layer, fluidity parameter, and concrete tensile strength.

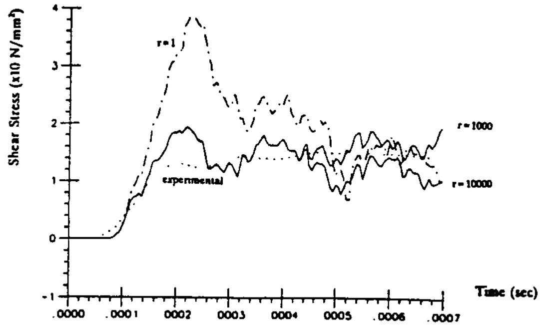


FIG. 24. Shear Stress at Loc 1 for Various r

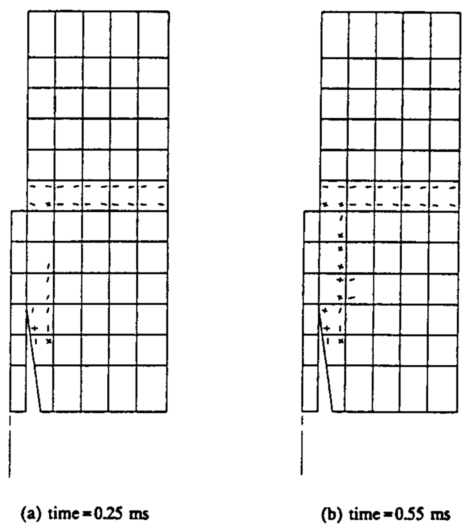


FIG. 25. Cracked Elements in Concrete

CONCLUDING REMARKS

The brief summary of results presented above indicates that the proposed analytical models show promise for predicting response of concrete structures to impact loads. Further research is required, however, to generalize the contact simulation to include sliding on the surface, to improve the material modeling, and to determine suitable criteria for optimum mesh size and time step. It is also clear

that appropriate modeling of the impacting mass is important. Improved modeling capabilities and test data would be useful for determining appropriate design criteria for impact loading.

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CONVERSION FACTORS

1 in. = 25.4 mm

1 in.^{2} = 645.2 mm^2

1 lb. = 4.448N

1 psi = 0.006895 MPa

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